SCS
NATIONAL
ENGINEERING
HANDBOOK

SECTION 8

ENGINEERING GEOLOGY

Chapter 1 DESCRIPTION OF MATERIALS

Chapter 2 EXPLORATION METHODS AND EQUIPMENT

Oct. 1970

SOIL CONSERVATION SERVICE
UNITED STATES DEPARTMENT OF AGRICULTURE

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SOIL CONSERVATION SERVICE
Washington, D. C. 20250

October 20, 1970

NATIONAL ENGINEERING HANDBOOK NOTICE 8-2

The following change should be made in the National Engineering Handbook, Section 8, Engineering Geology:

On page ~~1020~~ 1-20, the paragraph headed Benzidine Test should be deleted. This change is necessary because the use of benzidine may be hazardous to human health.

Kenneth E Grant

STC
EWP
WO
All Professional Engineers
and Geologists

PREFACE

The portion of the Handbook contained herein presents criteria, methods, and equipment for making geologic investigations of damsites and taking samples for laboratory analyses. It is for the use of geologists and engineers in the Soil Conservation Service.

Numerous individuals, both geologists and engineers, have contributed generously to the preparation of this section of the Handbook. Their contributions are herewith gratefully acknowledged.

Note: This issue is for in-Service use and material contained here is not released for publication.

Washington, D. C. 1961

Reprinted with minor revisions and corrections, April, 1968.

NATIONAL ENGINEERING HANDBOOK

SECTION 8

ENGINEERING GEOLOGY

Safety and Precautions

All safety practices and procedures currently established by safety handbooks and guides of the Soil Conservation Service must be adhered to in field operations.

Emphasis on safety measures as regards drill crews should be placed on the use of safety helmets and other protective devices such as gloves and hard-toed shoes. Personnel operating drill rigs or other persons whose duties require close proximity to machinery in operation or transit, should rid themselves of ragged or torn clothing. Machinery in operation should be equipped with guards on any moving parts insofar as practicable.

Equipment operators should not run the equipment in excess of the limits of capability and safety as established and designated by the manufacturer.

Equipment should not be presumed to be in safe operating condition unless it has been adequately checked by a competent, responsible person.

Regular condition checks should be made on all equipment and the results reported.

Caution must be used when operating equipment in the vicinity of power transmission lines. Consideration should be given to the possible presence of underground utility lines.

Wire ropes or cables used with truck winches frequently are broken. Drilling party personnel should stay well clear of the reach of the cable during operations of the winch.

Crews using geophysical instruments or making other investigations involving explosive charges should be well acquainted with the precautions necessary to avoid accidents. Only properly licensed blasters may handle, load and fire explosive charges. Until such time as this method of exploration is approved for full SCS use, and SCS regulations issued, it is recommended that locally employed, licensed blasters be used.

Where trench or pit excavations require side supports of cribbing, determine that the material for the cribbing is of adequate strength and is so installed that slumping, caving and sliding cannot occur.

Test holes should be covered each evening and plugged level with the surface upon completion of exploration, to prevent accidents. An open hole is a potential danger to humans and livestock; it could cause a broken leg or even more serious accidents. Test pits and trenches should be leveled also upon completion of site investigations.

Caution should be exercised in the handling of radioactive materials and caustic, toxic, or flammable chemicals; for example, the nitrobenzene and benzidine hypochlorite used in clay mineral tests is poisonous not only if taken internally, but also by absorption through the skin or by inhalation of the vapor.

Avoid personal accidents! Be sure to make reports on accidents within 24 hours. (See Administrative Procedures Handbook.) Get medical assistance if required, even if it is necessary to shut down operations.

All crews should have copies of the First Aid Guide and should have first aid kits as prescribed in the Guide. Snakebite kits are required in poisonous snake-infested areas.

Caution should be exercised when moving drilling equipment on roads, streets, and highways.

Bran or other grain derivatives never should be added to drilling mud since this mixture is detrimental to livestock.

Dye tracers to be used in ground water must be nontoxic to both humans and livestock. Determination of this factor must be made prior to use.

NATIONAL ENGINEERING HANDBOOK

SECTION 8

ENGINEERING GEOLOGY

Introduction

Purpose and Scope

The purpose of this part of the National Engineering Handbook is to present, in brief and usable form, information on equipment, tools, exploration and sampling techniques, and criteria for conducting adequate investigations of damsites. The material is compiled to assist technicians in planning site investigations, carrying out field investigations, and preparing reports within the framework of established SCS standards. The guide also will serve as a useful tool for training purposes and to promote extablishment of uniform standards and procedures for geologic investigations of damsites.

The choice of design and construction methods for a particular dam is contingent upon the characteristics of materials of which, and upon which, the structure is to be built. Knowledge of these materials, sufficient in scope and quality to satisfy design and construction requirements, is necessary for each site if consistent development of economically sound and practical structures is to be achieved. Such knowledge is acquired by thorough geologic examination of sites, by accurate foundation and borrow exploration, by soil mechanics laboratory tests, and thorough practical experience in a particular area.

This handbook is not intended to be a complete technical treatise on the subject of site investigations. Nor is it intended to establish a stereotyped pattern for site investigations. Each structure site has its own particular characteristics. The geologist and engineer must establish a pattern of investigations and application of exploration and sampling methods dependent on the site conditions, to obtain the information needed for design and construction. This requires sound judgment and a knowledge of requirements for design and construction as well as a knowledge of exploration and sampling techniques. The geologist must become thoroughly familiar with basic principles and techniques in the fields of engineering geology, soil mechanics, design, and construction, to achieve technical competence. He must work closely with the project, design, and construction engineers on each site in order to determine the requirements for that particular site and to establish investigational procedures.

Use of Earth Materials

The use of earth materials is stressed because they have three aspects of major importance: (1) The materials are usually present in abundance in the immediate area of the structure; (2) the materials have properties which permit their use for structural purposes; and (3) their availability allows a greater economy than manmade, imported materials.

Embankments

The use of natural materials in embankments is related to their inherent capacity to be remolded with, in some types, an accompanying modification in their engineering properties. Earth embankments may be homogeneous or zoned with the materials selected and remolded to form an essentially water-impervious barrier as well as a structurally strong unit.

Foundations

Materials composing foundations must support, without danger of failure, rupture, or displacement, the loads to be superimposed. The types of foundations range from bedrock to unconsolidated sediments and, as the types vary, so must the method and intensity of investigation vary.

Spillways

The spillway controls the rate of discharge through the structure. By virtue of its stability and resistance to erosion, the spillway insures the life of the entire structure. Earth spillways are susceptible to erosion. When they are to be constructed of, or in, erodible materials, great care should be given to the identification and classification of the materials.

Other Uses

Coarse-grained materials may be used for foundation and embankment drains and for erosion protection.

Riprap may be placed on the embankment so as to afford protection from the impact of water and weather.

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ENGINEERING GEOLOGY

CHAPTER 1 - DESCRIPTION OF MATERIALS

INTRODUCTION

General

The adequacy of a geologic investigation of a structure site depends upon accuracy in the description and classification of materials at the site and proper interpretation in respect to engineering requirements. Materials at a particular site are to be described and classified according to their geologic and physical properties and their engineering or behavior properties. These are necessary to establish correlation and stratigraphy of the site and to develop the design of the structure and construction methods to fit the particular site condition. This chapter outlines some of the more important properties of soil and rock materials which need to be considered in describing and classifying such materials.

Two systems of describing materials are employed in the engineering phases of damsite investigations: a geologic system, and the Unified Soil Classification System. The geologist must be familiar with both systems. The geologic system is based on the geologic and physical properties of materials. The Unified Soil Classification System is based on a combination of physical and behavior properties. To some extent the two systems overlap and descriptions developed for geologic interpretations also are used for engineering interpretations.

The successful engineering geologist must have a working knowledge of engineering design and construction methods in order to adequately describe and classify materials for engineering purposes. He must understand the terminology used by engineers and be able to present his interpretation in terms readily understood by them. It must be remembered that investigations of damsites are made to obtain information specifically for engineering purposes. Therefore, the terminology used in this handbook departs somewhat from standard terms normally found in geologic texts. The following is a list of some of the more important terms and their meanings as used in this handbook to describe materials:

Rock - A compact, semi-hard to hard, semi-indurated to indurated, consolidated mass of natural materials composed of a single mineral or combination of minerals.

Soils - Unconsolidated, unindurated, or slightly indurated, loosely compacted products of disintegration and decomposition.

Grain - A rock or mineral particle.

Gradation - Relative size distribution of particles.

Well graded - No sizes lacking or no excess of any size range, poorly sorted.

Poorly graded - Skip grades or excess of certain size ranges, may be well sorted.

Silt and Clay - Particles smaller than No. 200 mesh, identified by behavior characteristics rather than specific grain sizes.

Physical and Mineralogical Characteristics of Materials

Particle Characteristics

Particle characteristics, including size, shape, mineral composition, and hardness, are important considerations in establishing the origin of materials and geologic processes involved; and for determining the stratigraphy of the site. Lithologic similarity is one of the bases for establishing correlation and continuity of strata and equivalency in age. Particle characteristics also are important considerations in establishing the engineering properties and behavior characteristics of materials. The following briefly outlines some of the properties of particles and methods of classification for damsite investigations:

Size - The important classifications of size are: boulders, cobbles, gravel, sand, silt, and clay. Numerous grade scales have been developed to establish the limits of size for each of these classifications. The grade sizes used in the Unified Soil Classification System are to be used in the engineering geology phases of SCS work. Table 1-1 shows some of the commonly used grade scales for comparison.

Shape - Geologists express the degree of roundness of particles on the basis of the average radius of the corners divided by the radius of the maximum inscribed circle. Although particle shapes can be expressed numerically by this method, such a degree of accuracy is not required for geologic investigation of damsites. Visual estimation is sufficient for classification of equidimensional particles. Figure 1-1 shows a comparison of degrees of roundness and angularity which will serve as a guide to visual estimation and classification of roundness.

Table 1-1. Comparison of Grade Scales

in.	UNIFIED	AASHO	AGU	WENTWORTH	mm.
	boulders	coarse gravel or stone	very large boulders	boulder gravel	4026
			large boulders		2048
			medium boulders		1024
			small boulders		512
12	large cobbles		large cobbles	cobble gravel	256
6	small cobbles		small cobbles		128
3	coarse gravel		very coarse gravel	pebble gravel	64
1 3/4	fine gravel	medium gravel or stone	coarse gravel		32
3/8 mm.		fine gravel or stone	medium gravel		16
			fine gravel		8
4.76	No. 4* coarse sand		very fine gravel	granule gravel	4
2.00	No.10* medium sand	coarse sand	very coarse sand	very coarse sand	2
1.00			coarse sand	coarse sand	1
0.42	No.40* fine sand	fine sand	medium sand	medium sand	1/2
0.25			fine sand	fine sand	1/4
0.125			very fine sand	very fine sand	1/8
0.074	No.200* silt or clay	silt	coarse silt	silt	1/16
			medium silt		1/32
			fine silt		1/64
			very fine silt		1/128
		.005 mm. clay	coarse clay size	clay	1/256
			medium clay size		1/512
		colloids	fine clay size		1/1024
			very fine clay size		1/2048

*U.S. Standard Sieve Number

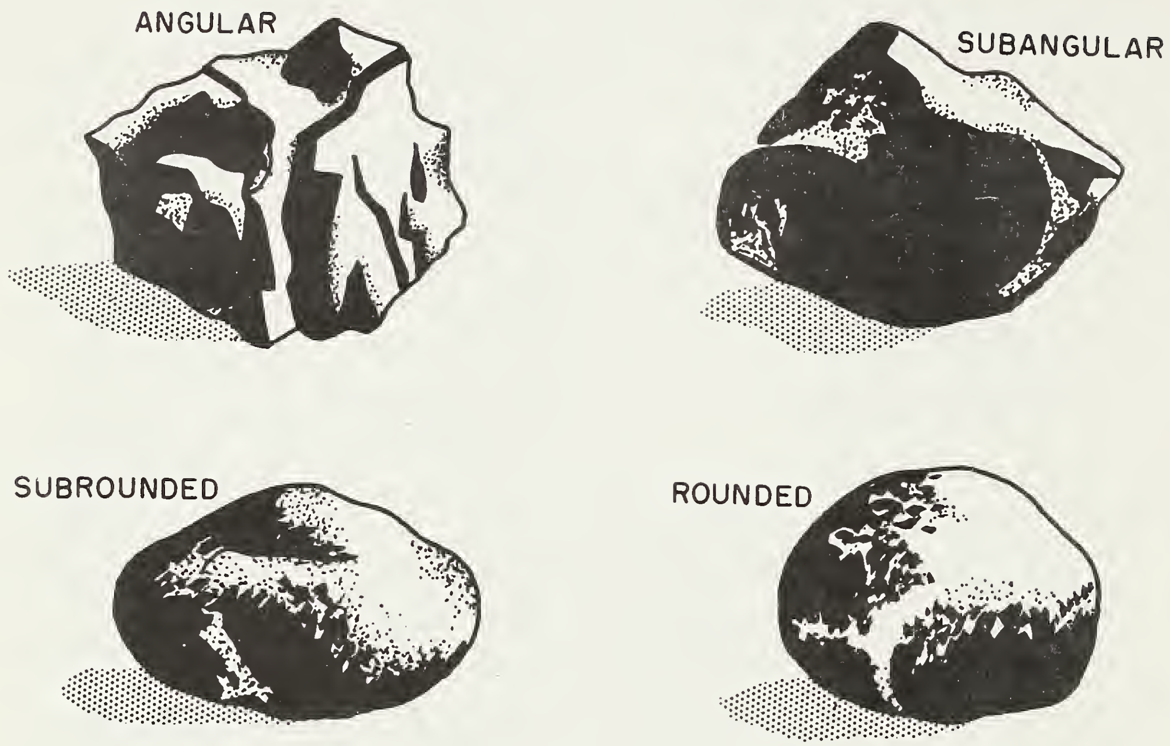

Figure 1-1 Particle Shapes

The above classification is adopted primarily to equidimensional particles of materials coarser than silt particles.

Normally it is not adequate for expression of non-equidimensional constituents either in coarse or fine-grained fractions of materials. Where flaky minerals are present these should be described on the basis of the mineral name instead of the shape, viz., biotite, muscovite, chlorite, etc. Where platy or prismatic rock fragments are present the rock type or structure controlling the shape such as bedding, cleavage, schistosity, etc., should be given as well as degree of rounding.

This classification is adopted primarily for equidimensional particles coarser than silt sizes. Normally, it is not adequate for expression of nonequidimensional constituents either in coarse or fine-grained fractions of materials. Where non-equidimensional minerals are present they should be described in such terms as platy, tabular, or prismatic. Where platy or prismatic rock fragments are present, the rock type or structure controlling the shape; such as bedding, cleavage, or schistosity; should be given as well as degree of roundness.

Mineral composition - The mineral composition of site materials varies greatly from place to place, depending upon the genesis of the materials and the geologic processes involved. The mineral composition may vary also with particle size at a particular site. The proportion of platy minerals usually increases over equidimensional minerals as the particle size decreases.

The coarse-grained materials are normally dominated by those rock-forming minerals, which are more resistant to chemical weathering, such as quartz and the heavy minerals. Rock fragments and unaltered rock-forming minerals, such as feldspar, calcite, and mica also may be present. The less complex minerals in the coarse-grained fractions can be identified readily by megascopic methods. Wherever this is possible, the predominant rock or mineral constituents and those rocks and minerals having a deleterious effect on engineering properties should be noted, using standard geologic terms.

The fine-grained materials represent the products of chemical and mechanical weathering. The mineral composition, together with weathering processes, controls the ultimate size and shape of the fine-grained particles. Quartz, feldspar, and many other minerals may, under mechanical weathering, be reduced to fine-grained equidimensional particles, such as in rock flour. Some types of minerals are broken down mechanically into platy particles. Micas are of this type. Alteration products of other types of minerals may result in the formation of platy particles.

Clay minerals - A group of minerals, known as clay minerals, requires special attention because of the influence of individual minerals on the engineering properties of soils. This is brought about by their inherently fine-grained nature, platy shape, and molecular structure. Clay minerals are predominantly hydrous aluminum silicates or more rarely, hydrous magnesium or iron silicates. Clay minerals are composed of layers of two types: (1) silicon and oxygen (silica layer) and (2) aluminum and oxygen or aluminum and hydroxyl ions (alumina or aluminum hydroxide layer).

There are three principal groups of clay minerals: kaolinites, montmorillonites, and illites. Because of variable influence of each type on the engineering property of soils, it is important

that the predominant clay mineral be properly identified whenever possible.

The kaolinite clays consist of two layer molecular sheets, one of silica and one of alumina. The sheets are firmly bonded together with no variation in distance between them. Consequently the sheets do not take up water. The kaolinite particle sizes are larger than those of either montmorillonite or illite and are more stable.

The montmorillonite clays consist of three layer molecular sheets consisting of two layers of silica to one of alumina. The molecular sheets are weakly bonded, permitting water and associated chemicals to enter between the sheets. As a result, they are subject to considerable expansion upon saturation and shrinkage upon drying. Particles of montmorillonite clay are extremely fine, appearing as fog under the high magnification of the electron microscope. Montmorillonite clays are very sticky and plastic when wet, and are of considerable concern in respect to problems of shear and consolidation.

Illite has the same molecular structure as montmorillonite but has better molecular bonding, resulting in less expansion and shrinkage properties. Illite particles are larger than montmorillonite and adhere to each other in aggregates.

Hardness - The hardness of individual minerals is normally expressed by geologists by means of the Mohs scale. Hardness, along with color, luster, transparency, streak, crystal form, cleavage or fracture, and specific gravity is an important property for identification of minerals. Hardness of individual particles is an important engineering consideration in respect to resistance to crushing when loaded.

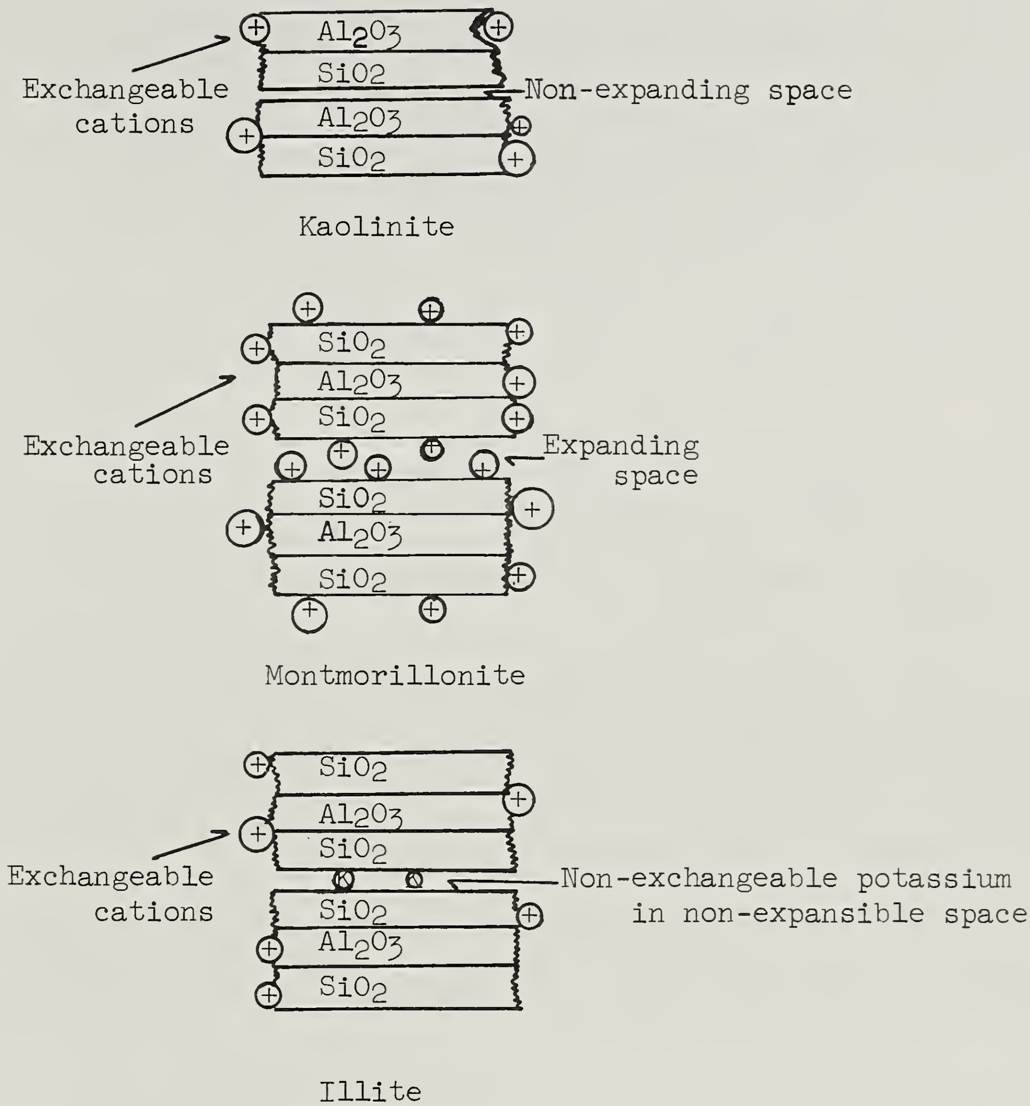

Figure 1-2. Representation of the Structure of Clay Mineral Particles

(From J. G. Cady, "Characteristics and Behavior of Soil Clay, SCS, 1954)

Mass Characteristics

Although individual particle characteristics are important for identification purposes and have an influence on engineering properties, associations of different particles impart mass characteristics and properties to both rock and soil materials, which are entirely different from those of the individual particles. This section briefly outlines mass characteristics which need to be described to develop adequate interpretations for geologic engineering purposes.

Soil materials - The term "soil materials" is here defined as the unconsolidated products of erosion and decomposition of rocks. It may include organic material. "Soil material" or "soil" consists of a heterogeneous accumulation of mineral grains, including most every type of uncemented or partially cemented inorganic and organic material to be found on the earth's surface. Soil materials may be referred to as cohesive or noncohesive, depending upon the tendency of the particles to adhere to one another.

Consistency - With increasing water content a solid clay mass changes consistency and passes from a solid state through a semisolid and plastic to a liquid state. The moisture contents, expressed in percent of dry weight, at which the mass passes from one of these stages of consistency to another are known as the Atterberg limits or limits of consistency.

The term consistency also is used to describe the relative ease with which a saturated cohesive soil can be deformed. In this sense, the consistency is described as very soft, soft, medium, stiff, very stiff, and hard.

The Atterberg limits or limits of consistency are determined on soil materials passing the 40-mesh sieve. The shrinkage limit or the limit between the solid and semisolid states is the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

The plastic limit is the water content corresponding to an arbitrary limit, fixed by a standard testing procedure, between the semisolid and plastic states of consistency.

The liquid limit is the water content corresponding to the arbitrary limit, fixed by a standard testing procedure, between the plastic and liquid states of consistency.

The plasticity index is a measure of the plastic state or the range of consistency within which a soil exhibits plastic properties and is numerically equal to the difference between the liquid limit and the plastic limit.

Density - The density or unit weight of a soil is defined as the weight per unit volume. The dry density is the weight of the unit mass excluding the weight of the contained water. The wet density includes the weight of the contained water.

Moisture content - The moisture content is the ratio of the weight of water contained in the soil to the dry weight of the soil solids. This value is expressed as a percent.

Density, or unit weight, and moisture values are highly significant in embankment construction. A certain density may be specified to which the soil is to be compacted, and the moisture content at the time of compaction is very important for many soils.

Permeability - The permeability of a soil is its capacity to transmit fluids under pressure. It may vary in different directions. The water flows through the voids between the soil grains. Therefore, the larger the size of the pores and their interconnections, the greater the flow of water. It may be seen that coarse-grained soils are more permeable than fine-grained soils. A well-graded soil, having a good distribution of particle size from large to very fine, is relatively less permeable than a poorly-graded soil of a comparable size because the finer grains fill the space between the larger particles.

Coefficient of permeability - The coefficient of permeability of a given soil is the volume of flow of water through a unit area, in unit time, under unit hydraulic gradient and at a standard temperature. Area is measured at right angles to the direction of flow. There are many permeability units in use. The more common ones are:

Meinzers Units = gallons/ft^2/day under unit hydraulic gradient.

Feet/day = ft^3/ft^2/day under unit hydraulic gradient.

Cm/second = cm^3/cm^2/sec. under unit hydraulic gradient.

Feet/year = ft^3/ft^2/year under unit hydraulic gradient.

Inches/hour = $inch^3/inch^2$/hour under unit hydraulic gradient.

All units are for a standard water temperature. For precise measurements, correction to this temperature must be made. Unit head or unit hydraulic gradient is a gradient of 1:1 or 100 percent.

These units are readily interchangeable by multiplying by the proper factor as indicated in table 1-2.

Table 1-2. Conversion factors for permeability units

From \ To	Meinzers Units	Feet/day	Cm/sec.	Feet/year	Inches/hour
Meinzers Units	1.0000	0.13368	4.7159×10^{-5}	48.8256	0.06684
Feet/day	7.4806	1.0000	3.5278×10^{-4}	365.2422	0.50000
Cm/sec.	2.12049×10^{4}	2.83464×10^{3}	1.0000	1.03530×10^{6}	1.41731×10^{3}
Feet/year	0.02048	2.7379×10^{-3}	9.6590×10^{-7}	1.0000	1.3689×10^{-3}
Inches/hour	14.9611	2.0000	7.0556×10^{-4}	730.4844	1.0000

Consolidation - Consolidation refers to the volume change of a soil under load. Normally fine-grained soils consolidate more than coarse-grained soils and poorly-graded soils consolidate more than well-graded soils. Density, plasticity, porosity, permeability, and organic content are important factors in determining the degree of compressibility.

Shearing strength - Shearing strength is the resistance of soil particles to sliding upon one another.

Gradation - The term gradation is used here to describe the grain size distribution of unconsolidated or soil materials in keeping with engineering terminology. For engineering purposes the fine fraction (200 mesh) is classified as silt or clay on the basis of plasticity rather than on grain-size diameter.

This system is not entirely adequate to define all of the physical characteristics needed for identification and correlation purposes. The system to be used for geologic purposes consists of classification based primarily on the relative proportions of gravel, sand, silt, and clay. These classifications are outlined in figure 1-3.

Texture - Texture is defined as the geometrical aspects of the component particles of a rock, including size, shape, and spatial arrangement. Texture is important for field identification purposes and for predicting behavior of rock under load. Although specific geologic terms such as "phaneritic" and "aphanitic" imply specific descriptions of igneous rock, simpler terms such as "coarse-grained" and "fine-grained" should be employed to be more understandable. It is often more important to describe the presence of mineral constituents, degree of cementation, conditions of weathering, fracture system, and other properties having an influence on engineering properties, than to identify the type of rock. The symbols contained in figure 1-3, "Standard SCS Geologic Symbols" constitute a coverage normally adequate for classifying and describing rocks.

Rock structure - The structure of rocks can usually be given in a few simple terms describing holes, cavities, joints, bedding planes, fractures, cleavage, schistosity and similar features. Use of terms such as "vesicular" or "vugs" should be avoided where possible and always defined when used. Rock structure is an important consideration in respect to the amount and direction of water movement.

Strength of rock - The strength of rock is influenced by the mineralogical composition, shape of grains, texture, crystallinity, stratification, lamination, and other factors. Secondary processes such as cementation and weathering have a profound influence on the strength of rock. The following classifications,

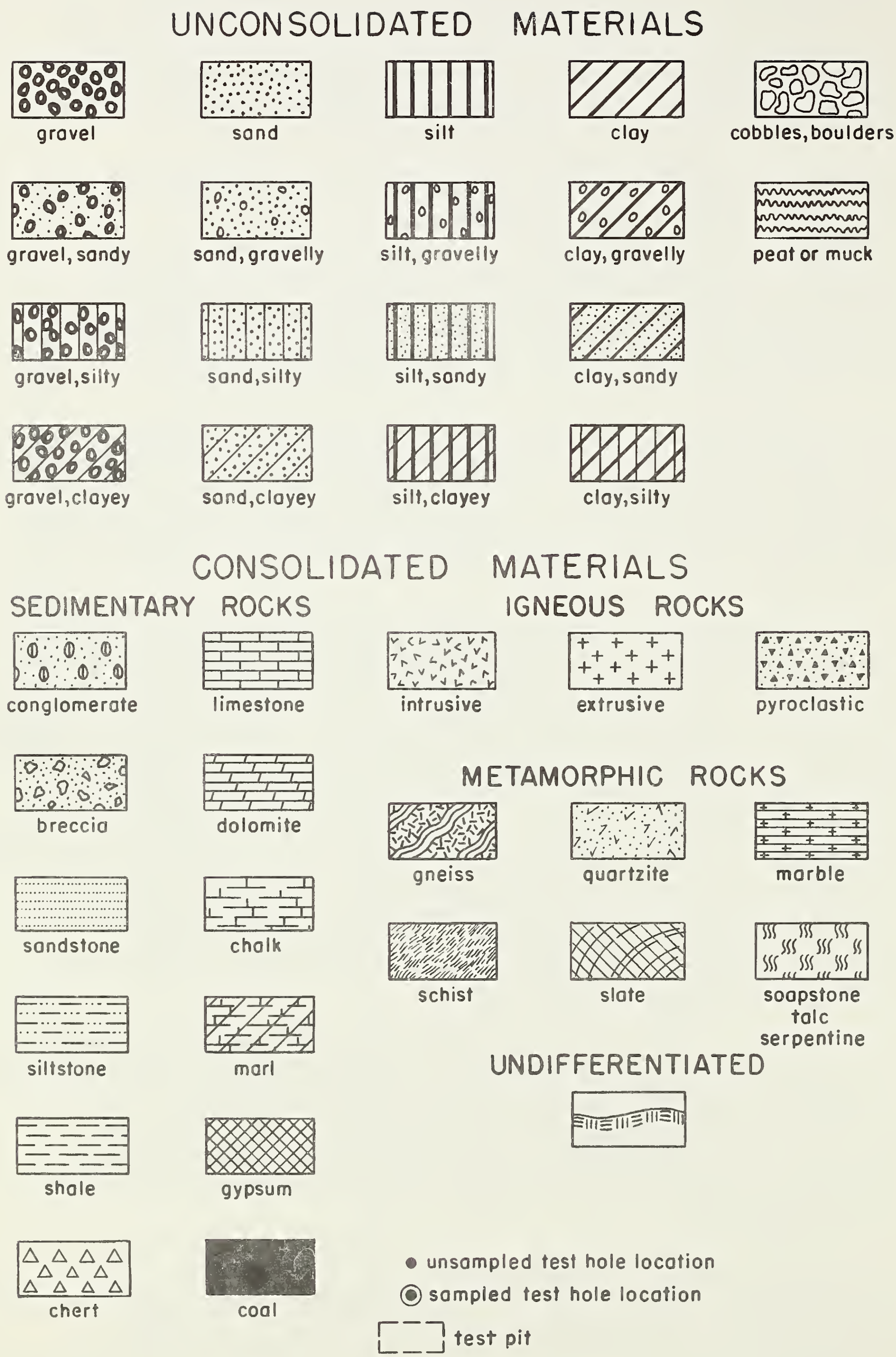

Figure 1-3. Standard SCS Geologic Symbols

based on field tests, are to be used for describing rock strength:

Very Soft - Permits denting by moderate pressure of the fingers.

Soft - Resists denting by the fingers, but can be abraded and pierced to a shallow depth by a pencil point.

Moderately Soft - Resists a pencil point, but can be scratched and cut with a knife blade.

Moderately Hard - Resistant to abrasion or cutting by a knife blade but can be easily dented or broken by light blows of a hammer.

Hard - Can be deformed or broken by repeated moderate hammer blows.

Very Hard - Can be broken only by heavy, and in some rocks, repeated hammer blows.

Cementation of rock is an important secondary process influencing the strength of rock. The principal cementing materials are silica, calcium carbonate, iron oxide, and clays. Most durable are bonds of silica, whereas clay bonds are weakest, particularly when saturated. It is important, therefore, to note the nature of cementing material when describing rock.

Chemical weathering occurs primarily through processes of hydration, oxidation, and carbonation. Chemical weathering not only influences strength of rocks but also the characteristics of soil materials derived therefrom. As a result of chemical weathering certain rocks break down into equidimensional grains whereas others break down into platy grains such as the clay minerals. Rocks, which contain minerals of variable resistance to chemical weathering, may become highly permeable through the alteration and removal of easily weathered materials and leaving the more resistant materials. The circulation of meteoric waters through fractured limestone and similar materials may develop solution channels of such large dimensions that collapse of foundations may be of concern. The products of chemical rock weathering have entirely different engineering properties than the rock from which they are derived. It is important, therefore, that the extent and character of these products be adequately described.

The strength of rock masses is greatly influenced by the presence of bedding, cleavage, schistosity, and similar features as well as by the presence of breaks such as joints and fractures. The spacing, pattern, attitude, and other characteristics of these features must be considered in evaluating strength of a rock mass. It is important that these characteristics be adequately described in describing rock masses.

Rock characteristics related to engineering properties - The character of rock is important from the standpoint of permeability, consolidation, shearing resistance, durability, and workability. The cost of structures may be greatly influenced by expensive rock excavation and by need for treatment of foundations, abutments, and reservoir basins. It is important, therefore, that where such problems exist, they be recognized and adequately described.

Foundations, abutments, and reservoir basins which are highly fractured, which contain solution channels, or are the products of differential weathering may be highly permeable. Practically all rocks have fractures. A rock mass, having extremely low porosity, might be highly permeable due to fractures and joints. Jointing is not restricted to any particular type of rock, but certain types of rocks in a particular area may possess a tendency toward larger and more closely spaced fractures than other types. Differential weathering may be found in many types of igneous and metamorphic rocks and certain sedimentary rocks. Differential weathering of cherty limestones, for example, may result in highly permeable rock foundations. It is important that the rate of permeability and the depth and direction of water movement be determined as closely as possible in order to determine requirements for foundation treatment. Field investigation may require angular boring, pressure testing, use of dyes or other tracer compounds, or other methods to properly determine permeability of rock.

The bearing strength of rock is normally adequate to support dams designed by the SCS. However, consolidation may be a problem in certain types of rock such as weakly cemented shales and siltstones, and rocks which have been altered to clay minerals. In each instance, samples of questionable materials are to be obtained for laboratory analysis, following the same procedures used for soil materials. Caverns or mines may present a problem of bearing or stability depending on the size and location of openings.

Problems of shear may result from poorly cemented shales and siltstones or highly weathered rock of low shear strength. Particular attention must be given to materials which dip in an adverse direction and which are subject to saturation or to unloading of their toe supports by excavation. This includes strata dipping downstream in foundations or strata dipping toward the centerline (parallel to the slope of the abutment) of proposed emergency spillway excavations. Rock strata of low shear strength must be thoroughly delineated and evaluated for design and construction purposes.

Cost of rock excavation may be greatly influenced by the nature of rock and secondary alteration. "Common excavation" and "rock excavation" are separate bid items for construction.

contracting. The geologist must describe rock proposed for excavation in terms translatable into workability by construction equipment so that the amounts of common and rock excavation can be determined in developing invitation for bids. For further details on classification of common and rock excavation, see SCS Standard Specifications, Construction and Construction Materials, Section 4-58, Item 4.2.

The following general descriptions of rock in terms of workability will prove helpful to geologists in describing rock proposed to be excavated and to contracting officers in interpreting descriptions for developing specific bid items. Very soft and soft rocks (See p. 1-13) can be excavated by power shovels or bulldozers in practically all cases, if the entire excavation is in the formation. Power shovels can excavate moderately soft formations and heavy power shovels can excavate moderately hard rocks. Moderately soft and moderately hard rocks have some degree of cementation and include partly cemented sandstones and marls and fairly compact shales. Most formations of hard rocks and those of very hard rocks must be removed by blasting if they have considerable bulk or thickness. Stratigraphy, attitude, and jointing are important factors in developing construction methods. Thus, thin beds of hard and very hard rocks may be removed by ripper, rock plow, or power shovel if they occur in beds of not more than 6 or 8 inches in thickness or are highly jointed. Hence, a series of soft shales interbedded with 6-inch layers of hard or very hard limestones usually can be removed with a power shovel. On the other hand, a massive bed of shale or crystalline gypsum 6 feet or more thick may require blasting, although neither would be rated higher than moderately soft.

Color - Color varies widely in materials but often provides a useful means of identification for geologic and engineering purposes. Thus, the presence of organic matter, certain minerals, and some types of weathering can often be readily detected by color. In classifying color of materials, care should be used to determine whether the coloring is due to inherent color of constituents, superficial stain or tarnish, or a weathering product. There may be a marked difference of color, depending on whether the material is dry or wet.

Geologic Properties of Materials

Stratigraphy

Stratigraphy deals with the formation, composition, thickness, sequence, and correlation of materials. Knowledge of the stratigraphy such as the continuity or discontinuity of certain beds or the distribution of critical horizons may be very important in interpreting site conditions.

Stratigraphy of the site is established from the study of particle and mass characteristics and the interpretation and extrapolation of the boring and test hole data. The determination of stratigraphy involves consideration of particle characteristics, their origin, mode of transportation (wind, water, ice, gravity) and the processes of deposition and consolidation. Guiding factors are the petrographic characteristics of the materials; e.g., mineral composition, size, shape, and spatial arrangement of the particles; and the type age, depth, thickness, sequence and continuity of the deposits.

Type of deposit - Type of deposit involves the mode, agent, and processes of formation of the deposit. It furnishes information on the continuity of strata and the uniformity of physical characteristics which may be encountered. For example, deposits of loess and glacial lake deposits (varved clays) may be remarkably consistent in thickness of strata and physical characteristics of materials. Other types, such as stream bar deposits, may pinch out in a matter of a few feet and the particle characteristics vary widely over short distances. It is important, therefore, that the type of deposit be accurately described in order to properly extrapolate continuity and physical characteristics of materials.

Standard geologic terms, simplified to the extent possible for adequate interpretations, should be used to describe the type of deposit. Such terms as granite, volcanic ash, marl, limestone, and gneiss, along with the formation name or age, are commonly used to describe rock materials. Because of the highly variable characteristics of sediments, however, a greater breakdown of terms which imply mode of origin should be employed. Such deposits should be described as fan, dune, colluvium, stream channel and other types denoting origin, in order to properly interpret physical characteristics.

Age - The age of a stratum establishes its vertical position in the geologic column and its relationship to other strata. Age should always be indicated using accepted geologic eras, periods, epochs, and ages when identifiable.

Depth, thickness, and continuity - The depth and thickness of materials at specific points at a site are determined from exposure and subsurface boring or test holes. Continuity must be interpreted on the basis of depth, thickness, type, and similarity of deposits and particle and bulk characteristics measured and described at different observation points. To facilitate interpretation of continuity, all measurements of depth should be referenced to a common elevation based either on mean sea level or an assumed datum plane. It is important that the vertical and areal continuity be determined for those materials which may have an effect on the design and construction of a dam. Continuity is best established on a graphic basis.

Depth and thickness of identified strata are to be plotted on graph paper at their proper elevations. Continuity lines are to be drawn in (dashed) where correlation of similar strata from different bore holes is possible. Forms SCS - 35A, 35B, 35C, and 35D, "Plan and Profiles for Geologic Investigation," are provided for this purpose. For examples, see chapter 4, figure 4-1. If a stratum in the vertical column of one observation cannot be correlated with any stratum in the next column, continuity has not been established. If correct interpretations have been made, the particular stratum is considered to be discontinuous. This should be shown by correlation lines which pinch out between bore holes. Discontinous strata are a common occurrence in types of materials having lenticular beds or where faults or other structural movements have resulted in shifting of beds to positions where they are not concordant. Whenever the limits of continuity cannot be established, and the discontinuity cannot be accounted for in the interpretations, additional observations are needed until sufficient exploration has been done to confirm lateral and longitudinal continuity or discontinuity.

Structure

The geologic structure of the site is of primary consideration in site selection. The term "structure" as applied to the geology of a damsite, refers to all of the geologic structural features either at the damsite or influencing the site. These features include faults, folds, unconformities, joints, fractures, rock cleavage, etc. Structure has an important influence on the geologic conditions of a site and the ultimate stability and safety of an engineering structure. Problems of leakage, sliding of embankments, uplift pressure in foundations, and differential settlement are often traced back to inadequate delineation and consideration of the geologic structure at the site.

Attitude - Attitude implies the geometric alinement of strata, faults, fractures, and other features, and is usually expressed in terms of dip and strike. In some instances, such as in plunging anticlines, for example, special conditions require more elaborate descriptions than dip and strike. In describing attitude, standard geologic terms should be used.

Folds - Folding is a common type of deformation in the earth's surface. Many folds extend over large areas so that deformation results in a more or less uniform dip and strike at a particular site. Smaller, local folds, however, are usually of more concern than those of a regional character. Minor folds which create channels with capacity for substantial subterranean water movement may escape detection in a geologic investigation of a damsite. Where such folds are suspected and anomalies of continuity in respect to apparent inclination of strata in bore holes are encountered, additional borings may be required to

determine the location and size of the folds for design considerations. Descriptions of folds should indicate their size, location, type (anticlinal, synclinal, drag) and the attitude of the limbs and axial plane.

Faults - A fault is defined as a break in the earth's crust along which movement has taken place. Displacement may be but a few inches or many miles. Faults may be detected by discontinuity of strata and by surface features. Aerial photographs often provide evidence of the presence of faults in an area. Faults may present a serious problem when they occur at a structural site. One that is active obviously presents a serious hazard. In addition, those that are now inactive may have so modified the geology that the site presents special problems in design, construction, or functioning of the proposed structure. Faults encountered at sites should be described in detail, including type, such as normal or reverse, attitude of the fault plane and the direction and amount of displacement. Juxtaposition of materials with quite different engineering properties and modification of ground water conditions are examples of the effects of faulting that may be important. Furthermore, the fault zone is of special importance if appreciable rock shattering or alteration of minerals has occurred, or if appreciable gouge has formed. In these instances, the approximate dimensions of the fault zone should be determined and changes in character of materials described.

Joints - Joints are defined as breaks in the rocks of the earth's crust along which no movement has occurred. Joints usually occur in systematic patterns. They may allow movement of ground water through otherwise impermeable material and this in turn may create problems in design, construction, or functioning of the structure. The number and orientation of joint systems and their spacing also influences the ease of rock excavation. Description of joints should include, besides their attitude, the spacing and the estimated depth of jointing, type of joints (strike, dip, or oblique,) and kind of joint system.

Paleontology

Evidences of life in the past are important for correlation purposes to establish continuity. Fossils are keys to correlation of rock strata. The presence of artifacts may be a means of distinguishing between Recent and Modern sediments. Plant and animal remains may have a very marked influence (usually adverse) on engineering properties. Thus, peat, muck, and carbonized plant remains have little value as construction materials. Tests or shells of foraminifera, algae, coral, and other components impart specific behavior characteristics to engineering materials. Descriptions of

artifacts and fossils, where they have little or no influence on the engineering properties of materials, should be limited to brief notes needed for correlation purposes. More detailed descriptions are needed where such materials have an influence on the engineering properties. These should include description of the nature of materials, including name, their extent and distribution in the formation.

Field Tests

The geologist may need to make field tests to further delineate geologic properties and to classify materials more accurately. The classification of unconsolidated materials for engineering purposes is done according to the Unified Soil Classification System, using standard tests developed for this purpose. These standard tests are described in the section on the Unified Soil Classification System. In addition to these standard tests, additional tests may be employed to aid in classifying materials and identifying special properties. Some of the tests are described below.

Acid test - Effervescence when a drop of dilute hydrochloric acid (one-tenth normal) is placed on a soil or rock indicates the presence of calcium carbonate.

Trailing fines - When a small sample of pulverized dry soil is shaken in the palm of the hand at a slight angle, the fine portion will trail behind. This is an aid in determining the relative proportion of the various grain sizes.

Shine test - When a dry or moist lump of soil is cut with a knife, a shiny surface indicates the presence of plastic clay.

Taste test - A dry lump of soil with a high clay content will adhere to the tongue.

Ribbon test - Plastic clays, when squeezed between the finger and thumb with a sliding motion, form a ribbon. The strength of the ribbon is an indication of the plasticity of the soil.

Odor - Organic soils have a pronounced and distinctive odor. Heating may intensify organic odors.

Acetone test - If gypsiferous soils are suspected, it may be necessary to conduct the following simple test:

1. Place 0.20 pound of air dry soil in a one-quart bottle and fill the bottle with distilled water.

2. Shake the soil-water mixture for about 20 minutes and then allow it to settle for 10 or more hours.

3. After this settling period, the solution above the soil will be clear if the soil contains significant amounts of gypsum. If the solution is cloudy, significant amounts of gypsum probably are not present.

4. Carefully pour about 1/2 ounce of the clear solution into a glass container without disturbing the settled soil in the bottom of the bottle.

5. Test this 1/2 ounce of solution for gypsum by adding 1/2 ounce of acetone to the solution. The presence of a milky, cloudy precipitate in the test solution indicates gypsum.

Benzidine test - A saturated water solution of the organic compound benzidine (or benzidine hydrochloride) produces a blue coloration in contact with clay minerals of the montmorillonite and illite groups, although the benzidine solution itself is slightly pink. The sample is not to be treated with hydrochloric acid before application of the stain. Manganese dioxide and organic materials may cause blue coloration in the absence of bentonite. Reducing agents (ferrous iron and others) sometimes prevent development of coloration. Gypsum greatly affects this test, but the influence may be cancelled out by first boiling the sample in water, pouring off the fine, suspended fraction, drying at 110°C., and finally staining.

Crystal-violet test - The crystal-violet staining solution causes montmorillonite to appear green at first and then change to a greenish yellow or orange yellow. The sample must be treated with hydrochloric acid prior to applying the stain. With this test illite attains a dark green color. Kaolinite merely absorbs the violet dye. The test solution consists of 25 cc of nitrobenzene and 0.1 gram of crystal violet.

Malachite-green test - Clay minerals of the kaolinite group (when treated with hydrochloric acid) show a bright apple-green color after application of malachite green solution. The solution consists of 25 cc of nitrobenzene and 0.1 gram malachite green. Montmorillonite and illite clays usually show greenish yellow or pale yellow.

Unified Soil Classification System

The Unified Soil Classification System provides a method of grouping unconsolidated earth materials according to their engineering properties. It is based on soil bahavior, which is a reflection of the physical properties of the soil and its constituents. This system has been accepted as a tentative standard by the ASTM and given designation D2487 (laboratory method) and D2488 (field methods).

For the purpose of classification, the system established 15 soil groups, each having distinctive engineering properties. Boundary classifications are provided for soils which have characteristics of two groups.

Letter symbols have been derived from terms which are descriptive of the soil components, gradation, and liquid limit. These are combined to identify each of the 15 soil groups. Table 1-3 lists these letter symbols.

Table 1-3. Letter Symbol

Component		Modifier	
Letter Symbol	Name	Letter Symbol	Name
None	Boulders or Cobbles	W	Well graded
G	Gravel	P	Poorly graded
S	Sand	M	Silty
M	Silt	C	Clayey
C	Clay	H	High liquid limit
O	Organic	L	Low liquid limit
Pt	Peat	---	---

Soil Components

The term "soil components" has been given to the solid mineral grains of which earth materials are composed. They range in size from over 12 inches average diameter to colloidal size. The particle size, gradation, shape, and mineral composition affect the behavior of the soil, as do the moisture content and the inclusion of other materials such as organic matter, gases, and coatings of cementing minerals. Table 1-4 lists various soil components with their associated grain sizes and descriptions and enumerates some of their significant properties. Comparison of grain size boundaries of the Unified System with those of other commonly used grade scales is shown in table 1-1.

Table 1-4

Soil Components and Significant Properties 1/

Soil Component	Symbol	Grain size range and description	Significant properties
Boulder	None	Rounded to angular, bulky, hard, rock particle, average diameter more than 12 in.	Boulders and cobbles are very stable components, used for fills, ballast, and to stabilize slopes (riprap). Because of size and weight, their occurrence in natural deposits tends to improve the stability of foundations. Angularity of particles increases stability.
Cobble	None	Rounded to angular, bulky, hard, rock particle, average diameter smaller than 12 in. but larger than 3 in.	
Gravel	G	Rounded to angular, bulky, hard, rock particle, passing 3-in. sieve (76.2 mm) retained on No. 4 sieve, (4.76 mm).	Gravel and sand have essentially same engineering properties differing mainly in degree. The No. 4 sieve is arbitrary division, and does not correspond to significant change in properties. They are easy to compact, little affected by moisture, not subject to frost action. Gravels are generally more pervious, stable, and resistant to erosion and piping than are sands. The well-graded sands and gravels are generally less pervious and more stable than those which are poorly graded. Irregularity of particles increases the stability slightly. Finer, uniform sand approaches the characteristics of silt; i.e., decrease in permeability and reduction in stability with increase in moisture.
Coarse		3 - 3/4 in.	
Fine		3/4 in. to No. 4 sieve (4.76 mm).	
Sand	S	Rounded to angular, bulky, hard, rock particle, passing No. 4 sieve (4.76 mm) retained on No. 200 sieve (0.074 mm).	
Coarse		No. 4 to 10 sieves: 4.76 to 2.0 mm.	
Medium		No. 10 to 40 sieves: 2.0 to 0.42 mm.	
Fine		No. 40 to 200 sieves: 0.42 to 0.074 mm.	
Silt	M	Particles smaller than No. 200 sieve (0.074 mm) identified by behavior; that is, slightly or nonplastic regardless of moisture and exhibits little or no strength when air dried.	Silt is inherently unstable, particularly when moisture is increased, with a tendency to become quick when saturated. It is relatively impervious, difficult to compact, highly susceptible to frost heave, easily erodible and subject to piping and boiling. Bulky grains reduce compressibility; flaky grains, i.e., mica, diatoms, increase compressibility, produce an "elastic" silt.
Clay	C	Particles smaller than No. 200 sieve (0.074 mm) identified by behavior; that is, it can be made to exhibit plastic properties within a certain range of moisture and exhibits considerable strength when air dried.	The distinguishing characteristic of clay is cohesion or cohesive strength, which increases with decrease in moisture. The permeability of clay is very low. It is difficult to compact when wet and impossible to drain by ordinary means, when compacted is resistant to erosion and piping, is subject to expansion and shrinkage with changes in moisture. The properties are influenced not only by the size and shape, (flat, plate-like particles), but also by their mineral composition; i.e., the type of clay-mineral, and chemical environment or base exchange capacity. In general, the montmorillonite clay minerals have greatest, and kaolinite the least adverse effect on the properties of soils.
Organic Matter	O	Organic matter in various sizes and stages of decomposition.	Organic matter present in even moderate amounts increases the compressibility and reduces the stability of the fine-grained components. It also may decay, causing voids, or by chemical alteration change the properties of a soil. Hence organic soils are not desirable for engineering uses.

1/ Adopted from Use of the Unified Soil Classification System by the Bureau of Reclamation, A. A. Wagner, Fourth International Conference on Soil Mechanics and Foundations, London, England, August 1957.

A 1/4-inch is approximately equivalent to the No. 4 U. S. Standard sieve. The No. 200 U. S. Standard sieve size is about the smallest particle visible to the naked eye. The No. 40 sieve size is the limit between medium and fine sand and "Atterberg limit" tests are performed on the fraction finer than the No. 40 sieve size in the laboratory.

Gradation

Well graded - Soils which have a wide range of particle sizes and a good representation of all particle sizes between the largest and the smallest present are said to be well graded.

Poorly graded - Soils in which most particles are about the same size or have a range of sizes with intermediate sizes missing (skip grades) are said to be poorly graded.

The gradation or grain-size distribution of soils consisting mainly of coarse grains is diagnostic of the physical properties of the soil. However, gradation is much less significant for predominantly fine-grained soils.

In the soil mechanics laboratory, the amounts of the various sized grains are determined by sieving and mechanical analysis and the results plotted on form SCS-353. The type of gradation is readily apparent from the shape of the grain-size curve. Figure 1-4 illustrates the grain-size distribution graphs of some typical soils. Poorly graded soils have steeply sloping curves, very flat curves, or abrupt changes in the slope of the curves, when plotted on semi-log graph paper. Well graded soils plot as smooth curves. To qualify as well-graded, the gradation must meet certain requirements in respect to coefficient of uniformity and coefficient of curvature of the plotted graph. The coefficient of uniformity (C_u), which is a measure of size range of a given sample, is the ratio of that size, of which 60 percent of the sample is finer (D_{60}); to that size, of which 10 percent of the sample is finer (D_{10}). The coefficient of the curvature (C_c), which defines the shape of the grain-size curve, is the ratio of the square of that size, of which 30 percent of the sample is finer (D_{30}), to the product of the D_{60} and D_{10} sizes. These ratios can be simply written:

$$C_u = \frac{D_{60}}{D_{10}} \qquad C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$$

See table 1-5 for explanation of the use of these coefficients and other criteria for laboratory identification procedures.

Consistency

The most conspicuous physical property of the fine-grained soils is their consistency which relates to plasticity or lack thereof.

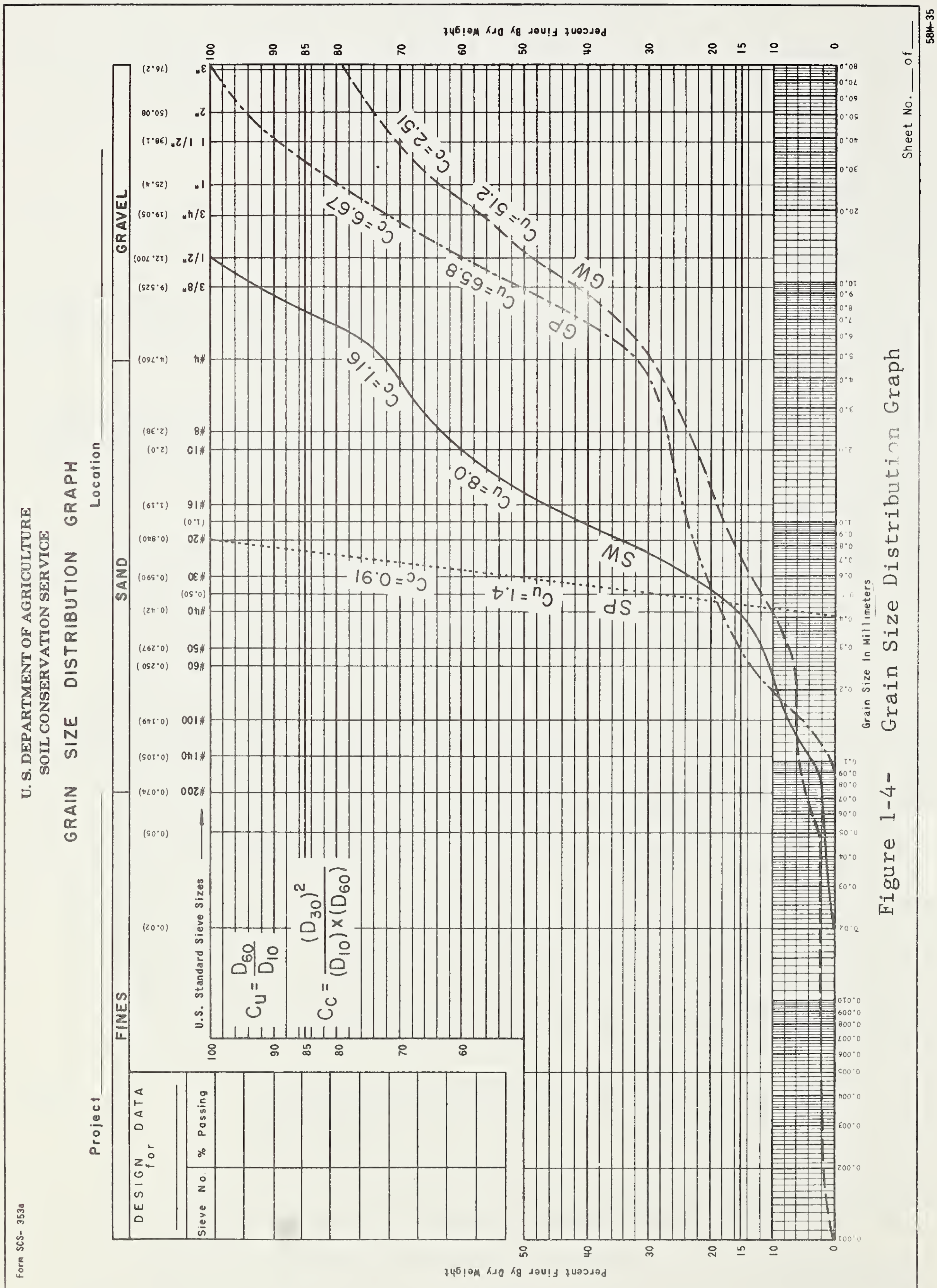

Figure 1-4- Grain Size Distribution Graph

The various stages of consistency have been described under Mass Characteristics. Atterberg tests are used to determine the liquid and plastic limits of soils in the laboratory. Field tests for dilatancy (reaction to shaking), dry strength (crushing characteristics), and toughness (consistency near the plastic limit) have been devised for field determinations. Tables 1-6 and 1-7 contain the procedures for making these field determinations and the methods of field classifications. The tests are illustrated in figure 1-5.

Field Classification Procedures

An adequate description of the soil material encountered in a geologic investigation is very important. Such characteristics as approximate percentage of the sizes, maximum size, shape, and hardness of coarse grains; mode of origin, and type of deposit; structure; cementation; dispersion; moisture and drainage conditions; organic content; color; plasticity; and degree of compaction; in addition to typical name and group symbol, should be recorded in accurate and precise but simple terms. Local or geologic names should be included also where possible.

The field procedure does not require specialized equipment. A supply of clear water in a syringe or oil can and small bottles of dilute hydrochloric acid, acetone, and benzidine and other reagents will facilitate the work. The geologist who lacks experience in classifying materials in the Unified Soil Classification System will find it expedient to use No. 4, No. 40, and No. 200 U. S. Standard Sieves in the field in the initial stages of training to aid in identifying relative quantities of coarse and fine-grained samples. Identification without the aid of sieves becomes relatively easy with practice and experience.

A representative sample is required for classification. The average size of the largest particle is estimated. The boulders and cobbles are removed and the percentage by weight in the total sample is recorded. The amount of over-sized material may be of importance in the selection of sources for embankment material. The distribution of boulders and cobbles and an estimate of their percentage in foundation materials should be noted so that their effect on the physical properties of the materials and possible construction problems can be evaluated. The rest of the procedure is, in effect, a process of simple elimination.

The following step-by-step procedure should be used:

1. Spread the sample on a flat surface or in the palm of the hand to aid in observing the relative amounts of coarse and fine-grained components. Classify the soil as coarse-grained

Table 1-5.--The Unified Soil Classification, Laboratory Criteria

Major division	Subdivision	Group	Note	Laboratory criteria		UNIFIED SOIL CLASSES
COARSE-GRAINED SOILS Less than half of material passes the No. 200 sieve size.	GRAVELS Less than half of the coarse fraction passes the No. 4 sieve size.	CLEAN GRAVELS Less than 5% passing the No. 200 sieve size.	Borderline cases require the use of dual symbols.	WELL GRADED Meets gradation requirements	GRADATION REQUIREMENTS ARE: $C_u = \frac{D_{60}}{D_{10}} > 4$ and, $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} <$ between 1 & 3	GW
				POORLY GRADED Does not meet gradation requirements		GP
		GRAVELS WITH FINES More than 12% passing the No. 200 sieve size.		Plasticity limits of material passing No. 40 sieve size plots below "A" line and P.I. less than 4.	Plasticity limits above "A" line with P.I. between 4 and 7 are border-line cases and require use of dual symbols	GM
				Plasticity limits of material passing No. 40 sieve size plots above "A" line or P.I. more than 7.		GC
	SANDS More than half of the coarse fraction passes the No. 4 sieve size.	CLEAN SANDS Less than 5% passing the No. 200 sieve size.	Borderline cases require the use of dual symbols.	WELL GRADED Meets gradation requirements	GRADATION REQUIREMENTS ARE: $C_u = \frac{D_{60}}{D_{10}} > 6$ and, $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} <$ between 1 & 3	SW
				POORLY GRADED Does not meet gradation requirements		SP
		SANDS WITH FINES More than 12% passing the No. 200 sieve size.		Plasticity limits of material passing No. 40 sieve size plots below "A" line and P.I. less than 4.	Plasticity limits above "A" line with P.I. between 4 and 7 are border-line cases and require use of dual symbols	SM
				Plasticity limits of material passing No. 40 sieve size plots above "A" line or P.I. more than 7.		SC
FINE-GRAINED SOILS More than half of material passes the No. 200 sieve size.	SILTS AND CLAYS Liquid limit less than 50	Below "A" line and P.I. less than 4		Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols		ML
		Above "A" line or P.I. more than 7				CL
		Below "A" line and P.I. less than 4 and $\frac{\text{L.L.(oven dry soil)}}{\text{L.L.(air dry soil)}} < 0.7$		PLASTICITY CHART (see below)		OL
	SILTS AND CLAYS Liquid limit greater than 50	Below "A" line				MH
		Above "A" line				CH
		Below "A" line and $\frac{\text{L.L.(oven dry soil)}}{\text{L.L.(air dry soil)}} < 0.7$				OH
HIGHLY ORGANIC SOILS				$\frac{\text{L.L. (oven dry soil)}}{\text{L.L. (air dry soil)}} < 0.7$		Pt

PLASTICITY CHART

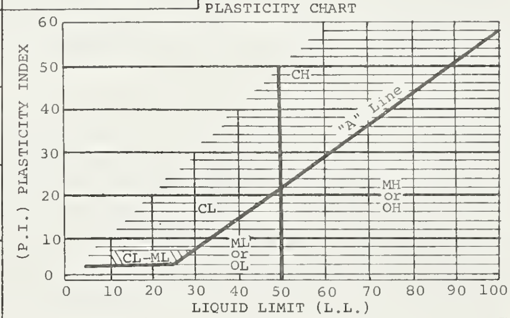

Table 1-6.--Unified Soil Classification, Field Identification

Major division		Subdivision		Type		UNIFIED SOIL CLASSES
COARSE GRAINED SOILS	More than half of material (by weight) is of individual grains visible to the naked eye.	GRAVEL AND GRAVELLY SOILS	More than half of Coarse Fraction (by weight) is larger than ¼ in. size.	CLEAN GRAVELS Will not leave a dirt stain on a wet palm.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	GW
					Predominantly one size or a range of sizes with some intermediate sizes missing.	GP
				DIRTY GRAVELS Will leave a dirt stain on a wet palm.	Nonplastic fines or fines with low plasticity (for identification of fines see characteristics of ML below).	GM
					Plastic fines (for identification of fines see characteristics of CL below).	GC
		SAND AND SANDY SOILS	More than half of Coarse Fraction (by weight) is smaller than ¼ in. size.	CLEAN SANDS Will not leave a dirt stain on a wet palm.	Wide range in grain size and substantial amounts of all intermediate particle sizes.	SW
					Predominantly one size or a range of sizes with some intermediate sizes missing.	SP
				DIRTY SANDS Will leave a dirt stain on a wet palm.	Nonplastic fines or fines with low plasticity (for identification of fines see characteristics of ML below).	SM
					Plastic fines (for identification of fines see characteristics of CL below).	SC

For visual classification the ¼ in. size may be used as equivalent to the No. 4 sieve size.

No. 200 sieve size is about the smallest particle visible to the naked eye.

Major division		Subdivision		ODOR	DRY CRUSHING STRENGTH	DILATANCY (SHAKE) REACTION	TOUGHNESS	RIBBON (NEAR THE P.L.)	SHINE (NEAR THE P.L.)	UNIFIED SOIL CLASSES
FINE GRAINED SOILS	More than half of material (by weight) is of individual grains not visible to the naked eye.	SILTS AND CLAYS (Low Plastic)	See Identification Procedures		Slight	Rapid	Low to None	None	Dull	ML
					High	Medium to None	Medium	Weak	Slight to Shiny	CL
				Pro-nounced	Medium	Slow to None	Low	None	Dull to Slight	OL
		SILTS AND CLAYS (Highly Plastic)			Medium	Very Slow to None	Medium	Weak	Slight	MH
					Very High	None	High	Strong	Shiny	CH
				Pro-nounced	High	None	Low to Medium	Weak	Dull to Slight	OH
HIGHLY ORGANIC SOILS					Readily identified by color, odor, spongy feel and frequently by fibrous texture.					Pt

Table 1-7.--Unified Soil Classification, Field Identification Procedures

<table>
<tr><th>FIELD IDENTIFICATION PROCEDURES
FOR FINE-GRAINED SOILS OR FRACTIONS</th><th colspan="2">INFORMATION REQUIRED DURING LOGGING</th><th>UNIFIED SOIL CLASSES</th></tr>
<tr><td rowspan="15">These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dry Strength (Crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.
High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.
Calcium carbonate or iron oxides may cause higher dry strength in dried material. If acid causes a fizzing reaction, calcium carbonate is present.

Dilatancy (Reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.
Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.
Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Toughness (Consistency near plastic limit)

After removing particles larger than No. 40 sieve size, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms, into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.
The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.
Highly organic clays have a very weak and spongy feel at the plastic limit.
Non-plastic soils cannot be rolled into a thread at any moisture content.
The toughness increases with the P.I.</td>
<td rowspan="8">COARSE
GRAINED
SOILS</td>
<td rowspan="8">For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.

Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.

Example:
<u>Silty sand</u>, gravelly; about 20% hard, angular gravel particles ½ in. maximum size; rounded and subangular sand grains coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).</td>
<td>GW</td></tr>
<tr><td>GP</td></tr>
<tr><td>GM</td></tr>
<tr><td>GC</td></tr>
<tr><td>SW</td></tr>
<tr><td>SP</td></tr>
<tr><td>SM</td></tr>
<tr><td>SC</td></tr>
<tr><td rowspan="6">FINE
GRAINED
SOILS</td>
<td rowspan="7">Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses.

For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.

Example:
<u>Clayey silt</u>, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess, (ML).</td>
<td>ML</td></tr>
<tr><td>CL</td></tr>
<tr><td>OL</td></tr>
<tr><td>MH</td></tr>
<tr><td>CH</td></tr>
<tr><td>OH</td></tr>
<tr><td>ORGANIC
SOILS</td><td>Pt</td></tr>
</table>

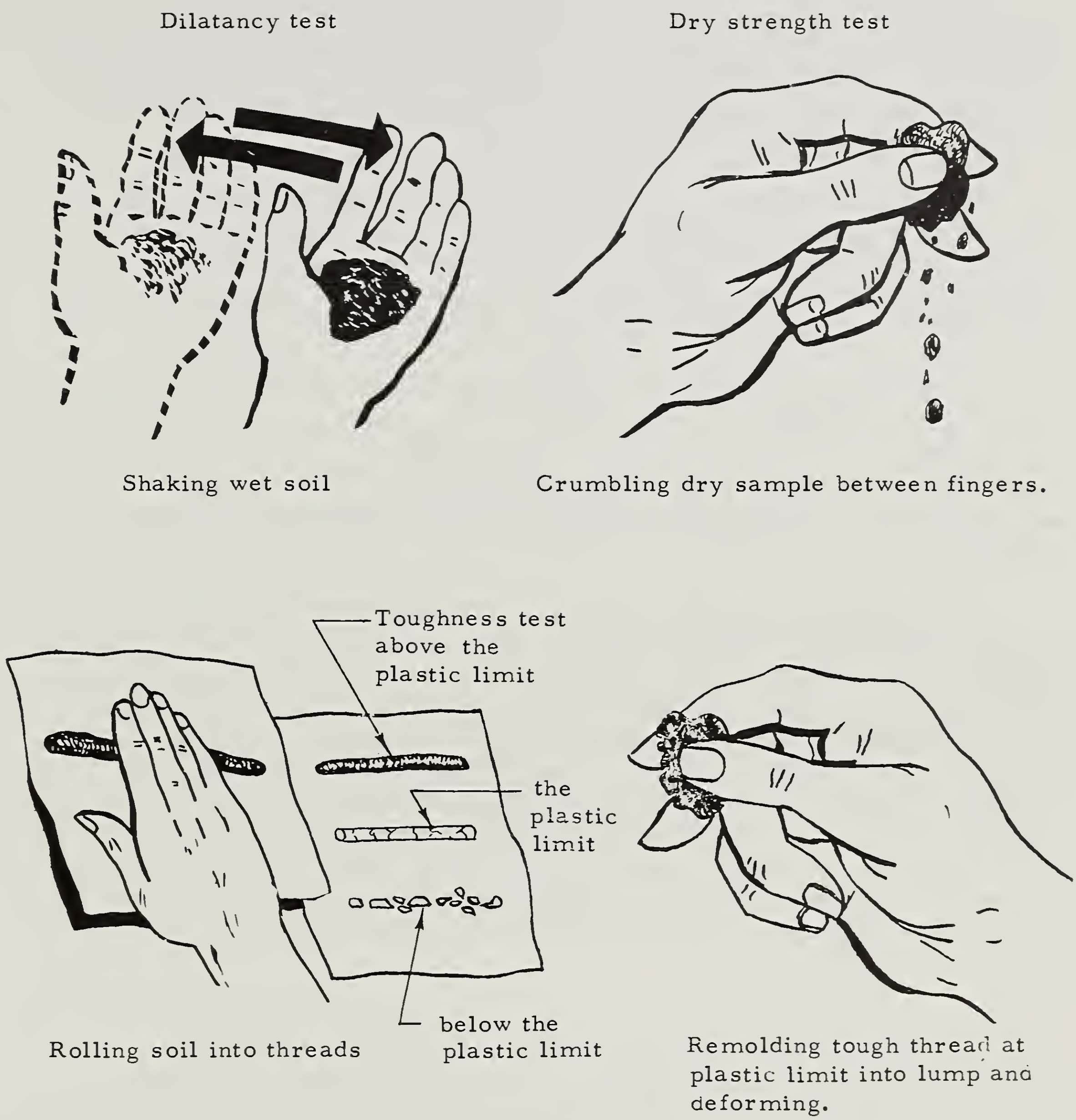

Figure 1-5.--Field Tests

or fine-grained. The division between coarse and fine grain is the 200 mesh sieve.

2. If fine-grained, see step 6 below. If coarse-grained, classify as gravel or sand, i.e., classify as gravel if more than 50 percent of coarse fraction is larger than No. 4 sieve (about 1/4 inch) and sand, if more than 50 percent of coarse fraction is smaller than No. 4 sieve.

3. If gravel or sand, determine whether it is "clean" having less than 5 percent fines; borderline, having 5 to 12 percent fines; or "dirty" having more than 12 percent fines. Fines are defined as the fraction smaller than the 200 mesh sieve size. Less than 5 percent fines will not stain the hands when wet.

4. If the gravel or sand is clean, decide if it is well graded (W) or poorly graded (P) and assign an appropriate group name and symbol: GW, GP, SW, or SP. Well graded materials have a good representation of all particle sizes. Poorly graded materials have an excess or absence of intermediate particle sizes.

5. If the gravel or sand contains more than 12 percent fines, it is classified as GM, GC, SM, or SC, depending upon the type of fines. The procedure for identifying type of fines is given in the following steps: Borderline cases, where fines range from 5 to 12 percent, are classified in the laboratory with dual symbols, i.e., GP-GC, SP-SC. Classification of borderline cases, as well as boundary cases between various groups, require precise laboratory analysis for proper classification. Such analyses cannot be made in the field. When field classification indicates that material might fall into one of two classifications, both symbols should be indicated, such as (GP or GC) or (SW or SP).

6. For fine-grained soils or the fine-grained fraction of a coarse-grained soil, the "dilatancy," "dry strength," and "toughness" tests are performed in accordance with the instructions given on the left-hand side of table 1-7. The group name and symbol are arrived at by selection of that group, the characteristics of which most nearly compare to that of the sample. These characteristics are shown in the lower part of table 1-6.

7. Highly organic soils are classified as peat (Pt). These are identified by color, odor, spongy feel and fibrous texture.

8. Fine-grained soils which have characteristics of two groups, either because of percentage of the coarse-grained components or plasticity characteristics, are given boundary classifications in the same way as coarse-grained soils. Boundary classifications which are common for fine-grained soils are (ML or MH), (CL or CH),

(OL or OH), (CL or ML), (MH or CH). Common boundary classifications between coarse and fine grained soils are (SM or ML) and (SC or CL).

9. Miscellaneous tests and criteria may be used to identify the occurrence of other substances and constituents. Some of these are outlined under Field Tests, pages 1-20 to 1-21.

Table 1-6, Field Identification Criteria, lists in tabular form the classification characteristics of the soil groups. The engineering geologist can only estimate the primary constituents of unconsolidated material in the Unified Soil Classification System. More exact mechanical analyses must be made in the laboratory. However, when the laboratory analyses become available, they should be compared with the original field estimates. In this way the geologist can improve the accuracy of his estimates.

Tables 1-8, 1-9, and 1-10, Engineering Properties of Unified Soil Classes, presents a general evaluation of the engineering properties of the various classes. They provide guidance in determining the suitability of a given soil for various engineering purposes.

Table 1-8.--Engineering Properties of Unified Soil Classes

TYPICAL NAMES	IMPORTANT PROPERTIES						UNIFIED SOIL CLASSES
	SHEAR STRENGTH	COMPRESS-IBILITY	WORKABILITY AS CONSTRUCTION MATERIAL	PERMEABILITY			
				WHEN COMPACTED	K CM. PER SEC.	K FT. PER DAY	
Well graded gravels, gravel-sand mixtures, little or no fines.	Excellent	Negligible	Excellent	Pervious	$K > 10^{-2}$	$K > 30$	GW
Poorly graded gravels, gravel-sand mixtures, little or no fines.	Good	Negligible	Good	Very Pervious	$K > 10^{-2}$	$K > 30$	GP
Silty gravels, gravel-sand-silt mixtures.	Good to Fair	Negligible	Good	Semi-Pervious to Impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	GM
Clayey gravels, gravel-sand-clay mixtures.	Good	Very Low	Good	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	GC
Well graded sands, gravelly sands, little or no fines.	Excellent	Negligible	Excellent	Pervious	$K > 10^{-3}$	$K > 3$	SW
Poorly graded sands, gravelly sands, little or no fines.	Good	Very Low	Fair	Pervious	$K > 10^{-3}$	$K > 3$	SP
Silty sands, sand-silt mixtures.	Good to Fair	Low	Fair	Semi-Pervious to Impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	SM
Clayey sands, sand-clay mixtures.	Good to Fair	Low	Good	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	SC
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	Fair	Medium to High	Fair	Semi-Pervious to Impervious	$K = 10^{-3}$ to 10^{-6}	$K = 3$ to 3×10^{-3}	ML
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Fair	Medium	Good to Fair	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	CL
Organic silts and organic silty clays of low plasticity.	Poor	Medium	Fair	Semi-Pervious to Impervious	$K = 10^{-4}$ to 10^{-6}	$K = 3 \times 10^{-1}$ to 3×10^{-3}	OL
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Fair to Poor	High	Poor	Semi-Pervious to Impervious	$K = 10^{-4}$ to 10^{-6}	$K = 3 \times 10^{-1}$ to 3×10^{-3}	MH
Inorganic clays of high plasticity, fat clays.	Poor	High to Very High	Poor	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	CH
Organic clays of medium to high plasticity, organic silts.	Poor	High	Poor	Impervious	$K = 10^{-6}$ to 10^{-8}	$K = 3 \times 10^{-3}$ to 3×10^{-5}	OH
Peat and other highly organic soils.	NOT SUITABLE FOR CONSTRUCTION						Pt

Table 1-9.--Engineering Properties of Unified Soil Classes for Embankments

EMBANKMENTS								
COMPACTION CHARACTER-ISTICS	STANDARD PROCTER UNIT DENSITY LBS. PER CU. FT.	TYPE OF ROLLER DESIRABLE	RELATIVE CHARACTERISTICS PERME-ABILITY	RELATIVE CHARACTERISTICS COMPRESS-IBILITY	RESISTANCE TO PIPING	ABILITY TO TAKE PLASTIC DEFORMATION UNDER LOAD WITHOUT SHEARING	GENERAL DESCRIPTION & USE	UNIFIED SOIL CLASSES
Good	125-135	crawler tractor or steel wheeled & vibratory	High	Very Slight	Good	None	Very stable, pervious shells of dikes and dams.	GW
.Good	115-125	crawler tractor or steel wheeled & vibratory	High	Very Slight	Good	None	Reasonably stable, pervious shells of dikes and dams.	GP
Good with close control	120-135	rubber-tired or sheepsfoot	Medium	Slight	Poor	Poor	Reasonably stable, not well suited to shells but may be used for impervious cores or blankets.	GM
Good	115-130	sheepsfoot or rubber-tired	Low	Slight	Good	Fair	Fairly stable, may be used for impervious core.	GC
Good	110-130	crawler tractor & vibratory or steel wheeled	High	Very Slight	Fair	None	Very stable, pervious sections, slope protection required.	SW
Good	100-120	crawler tractor & vibratory or steel wheeled	High	Very Slight	Fair to Poor	None	Reasonably stable, may be used in dike with flat slopes.	SP
Good with close control	110-125	rubber-tired or sheepsfoot	Medium	Slight	Poor to Very Poor	Poor	Fairly stable, not well suited to shells, but may be used for impervious cores or dikes.	SM
Good	105-125	sheepsfoot or rubber-tired	Low	Slight	Good	Fair	Fairly stable, use for impervious core for flood control structures.	SC
Good to Poor Close control essential	95-120	sheepsfoot	Medium	Medium	Poor to Very Poor	*Very Poor	Poor stability, may be used for embankments with proper control. *Varies with water content.	ML
Fair to Good	95-120	sheepsfoot	Low	Medium	Good to Fair	Good to Poor	Stable, impervious cores and blankets.	CL
Fair to Poor	80-100	sheepsfoot	Medium to Low	Medium to High	Good to Poor	Fair	Not suitable for embankments.	OL
Poor to Very Poor	70-95	sheepsfoot	Medium to Low	Very High	Good to Poor	Good	Poor stability, core of hydraulic fill dam, not desirable in rolled fill construction.	MH
Fair to Poor	75-105	sheepsfoot	Low	High	Excellent	Excellent	Fair stability with flat slopes, thin cores, blanket & dike sections.	CH
Poor to Very Poor	65-100	sheepsfoot	Medium to Low	Very High	Good to Poor	Good	Not suitable for embankments.	OH
DO NOT USE FOR EMBANKMENT CONSTRUCTION								Pt

Table 1-10.--Engineering Properties of Unified Soil Classes for Foundations and Channels

CHANNELS — LONG DURATION TO CONSTANT FLOWS.		FOUNDATION — FOUNDATION SOILS, BEING UNDISTURBED, ARE INFLUENCED TO A GREAT DEGREE BY THEIR GEOLOGIC ORIGIN. JUDGEMENT AND TESTING MUST BE USED IN ADDITION TO THESE GENERALIZATIONS.					
RELATIVE DESIRABILITY			RELATIVE DESIRABILITY		REQUIREMENTS FOR SEEPAGE CONTROL		UNIFIED SOIL CLASSES
EROSION RESISTANCE	COMPACTED EARTH LINING	BEARING VALUE	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	PERMANENT RESERVOIR	FLOODWATER RETARDING	
1	-	Good	-	1	Positive cutoff or blanket	Control only within volume acceptable plus pressure relief if required.	GW
2	-	Good	-	3	Positive cutoff or blanket	Control only within volume acceptable plus pressure relief if required.	GP
4	4	Good	2	4	Core trench to none	None	GM
3	1	Good	1	6	None	None	GC
6	-	Good	-	2	Positive cutoff or upstream blanket & toe drains or wells.	Control only within volume acceptable plus pressure relief if required.	SW
7 if gravelly	-	Good to Poor depending upon density	-	5	Positive cutoff or upstream blanket & toe drains or wells.	Control only within volume acceptable plus pressure relief if required	SP
8 if gravelly	5 erosion critical	Good to Poor depending upon density	4	7	Upstream blanket & toe drains or wells	Sufficient control to prevent dangerous seepage piping.	SM
5	2	Good to Poor	3	8	None	None	SC
-	6 erosion critical	Very Poor, susceptible to liquefication	6, if saturated or pre-wetted	9	Positive cutoff or upstream blanket & toe drains or wells.	Sufficient control to prevent dangerous seepage piping.	ML
9	3	Good to Poor	5	10	None	None	CL
-	7 erosion critical	Fair to Poor, may have excessive settlement	7	11	None	None	OL
-	-	Poor	8	12	None	None	MH
10	8 volume change critical	Fair to Poor	9	13	None	None	CH
-	-	Very Poor	10	14	None	None	OH
-	-	REMOVE FROM FOUNDATION					Pt

No. 1 is best numerical rating.

NATIONAL ENGINEERING HANDBOOK
SECTION 8
ENGINEERING GEOLOGY

CHAPTER 2 - EXPLORATION METHODS AND EQUIPMENT

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NATIONAL ENGINEERING HANDBOOK

SECTION 8

ENGINEERING GEOLOGY

CHAPTER 2. EXPLORATION METHODS AND EQUIPMENT

Introduction

This chapter outlines briefly the various methods of subsurface exploration applicable to SCS work and describes types of cutting and sampling tools and other equipment needed to conduct geologic investigations of dam sites.

Exposed Profiles

Natural Exposures

A complete investigation of formations in natural exposures at the surface is necessary to provide a basis for subsurface investigations and to eliminate unnecessary drilling. Natural exposures, when described in detail, serve the same purpose as other logs in establishing stratigraphy and other geologic conditions. A fresh surface is required for the preparation of adequate descriptions. An ordinary hand shovel or geologist's pick may be required for preparing the surface of a natural exposure.

Trenching and Test Pitting

General - Trenching and test pitting are simple methods of shallow exploration of easily excavated rock or soil materials which permit visual inspection of strata. This is of great value in logging profiles and selecting samples. If bedrock is anticipated at a shallow depth, trenches and test pits should be located on the centerline of the structure and dug parallel with it.

If bedrock is not at shallow depths, deep trenches or test pits should be offset from the centerline to avoid damaging the foundation of the structure. Shallow trenches or test pits may be dug adjacent to the centerline for correlation purposes.

In cases where pits or trenches penetrate or pass through materials which will constitute the foundation, it is a requisite that backfilling be performed in such a manner as to obtain soil densities (compaction) at least equal to the density of the original, in-place material. It is recognized that certain limitations exist in the use of trenching and test-pit excavating equipment for compacting fill material. However, every practical effort should be made to re-establish the in-place densities of foundation materials.

Trenches - Trenches imply long, narrow excavations. They are advantageous for studying the various formations on steep slopes and in exposed faces. Trenches made by power equipment, such as backhoes, draglines, and bulldozers, may require hand trimming of the sides and bottom to reach relatively undisturbed material.

The method is of particular value in delineating the rock surface beneath the principal spillway and in abutments and in exploring emergency spillway materials. In materials containing many cobbles or boulders, where drilling is difficult, trenching may be the most feasible method of investigation. On the center-line of the dam, trenches may yield valuable information on rock excavation and core trench depth, especially where thin-bedded or flaggy rocks are found near the surface.

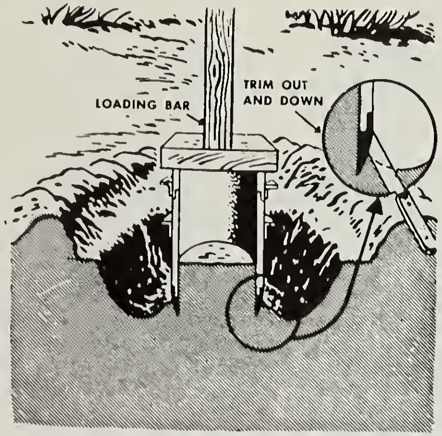
Test pits - Test pits are rectangular or circular excavations large enough to admit a man with sampling equipment. They may be excavated by hand or by the use of power equipment such as a clamshell or orange-peel bucket. Power equipment should be used only for rough excavation and with extreme caution when approaching the depths at which undisturbed samples are to be taken. Cribbing is required in unstable ground and for deep pits.

The advantages of test pitting are about the same as for trenching, but have the added advantage of being adaptable to greater depths at less cost. Disadvantages include time loss and cost where cribbing is necessary. With adequate dewatering equipment, they can be extended below the water table and with cribbing they can penetrate unstable materials.

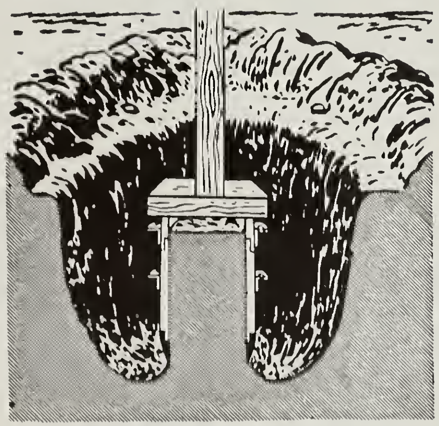
Procedures for Obtaining Undisturbed Samples from Exposed Profiles
Undisturbed hand-cut samples can be obtained above the water table from nearly all types of materials with less disturbance than by other means.

Undisturbed samples may be obtained as box, cylinder, or chunk samples. Box samples are hand-cut and trimmed to cubical dimensions and sealed in individual boxes for handling and shipping. They should have a minimum dimension of six inches. Cylinder samples from four to eight inches in diameter and six to twelve inches long can also be hand-cut by sliding a cylinder over a column of soil which is trimmed to approximate size in advance of the cylinder. Cylinder samples may also be obtained by jacking or otherwise pushing this-wall drive samples into exposed surfaces using a continuous steady pressure. Chunk samples are of random size and shape and are broken away from the soil mass with or without trimming. They are difficult to package and ship but are simple to obtain.

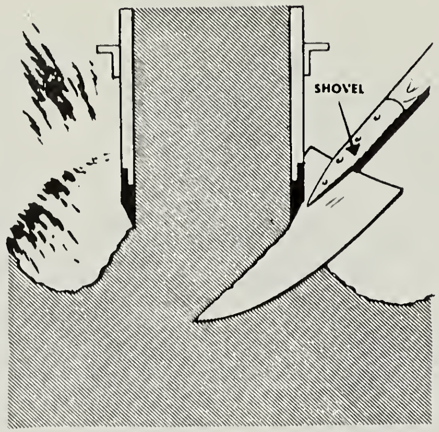
Figures 2-1, 2-2, and 2-3 demonstrate the methods of obtaining and packaging hand-cut samples.



Press mold down over soil firmly, using loading bar if necessary. Carefully trim soil away from sampling collar with knife. Cut downward and outward to avoid cutting into sample. The actual cutting to size is done with sampling collar.



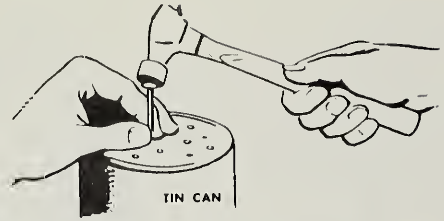
Excavate trench deeper and repeat process until soil penetrates well into extension collar.



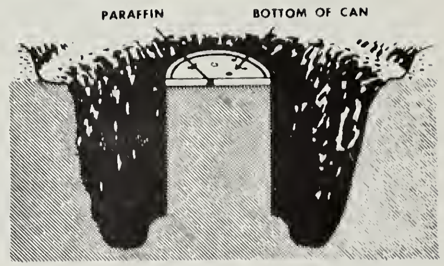
Cut off sample at bottom of mold with shovel, butcher knife, or wire saw and remove from hole.

Remove upper collar and trim top surface of sample; then turn mold upside down, remove sampling collar, and trim bottom. The top and bottom surfaces must be trimmed level with ends of mold. Protect ends with wood discs and tape around edges.

NOTE: If stones interfere, pick them out carefully and backfill with soil. Record this fact in log of sample whenever this is done.



1. Punch holes in bottom of can.



2. Excavate around can in the same way as for compaction mold and press down until soil penetrates to bottom of can.

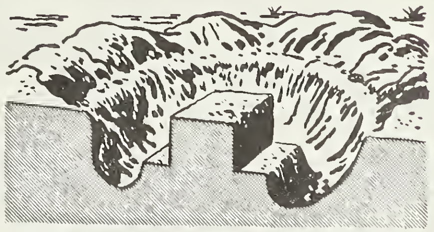
3. Pour paraffin in holes and seal bottom of can. Cut off sample with butcher knife and remove from hole.

4. Cut surface about ½-inch below top of can and fill with paraffin.

Figure 2-1 Methods of Obtaining Cylinder Samples

To obtain a chunk sample from a subgrade or other level surface such as the bottom of a test pit:

1. Smooth ground surface and mark outline of chunk.
2. Excavate trench around chunk.



3. Deepen excavation and trim sides of chunk with butcher knife.

4. Cut off chunk with butcher knife, trowel, or hacksaw blade and carefully remove from hole.

To obtain a chunk sample from the vertical face of a test pit or shovel cut:

1. Carefully smooth face surface and mark outline of chunk.

2. Excavate around and in back of chunk. Shape chunk roughly with butcher knife.

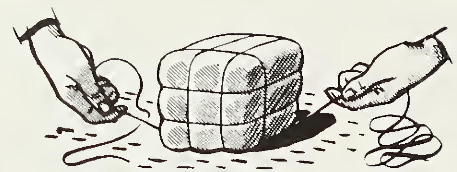
3. Cut off chunk and carefully remove from hole.

To seal chunk after removing it from hole:

1. Trim and shape rough edges with butcher knife.

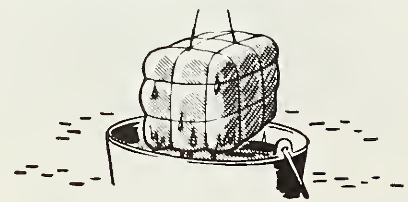
2. Apply three coats of paraffin with paint brush. Allow each coat to cool and become firm before applying next coat.

NOTE: This gives adequate protection for strong samples that are to be used within a few days. Samples that are weak or may not be used soon require additional protection.



3. Wrap with cheesecloth or other soft cloth. If cloth is not available, reinforce with several loops of friction tape or twine.

4. Apply three more coats of paraffin.



NOTE: A better method is to dip entire sample in melted paraffin after first brush coat is applied. This requires a large container and more paraffin, but gives a more uniform coating. By repeated dipping, paraffin can be built up to a minimum $\frac{1}{8}$-inch thickness.

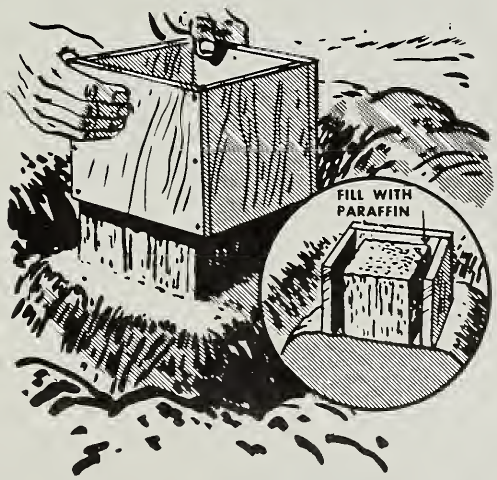
Figure 2-2. Chunk Samples

Box Samples

Box samples are sometimes used for large undisturbed samples requiring extensive investigation. They can be firmly packed for shipment or storage, but require considerable paraffin.

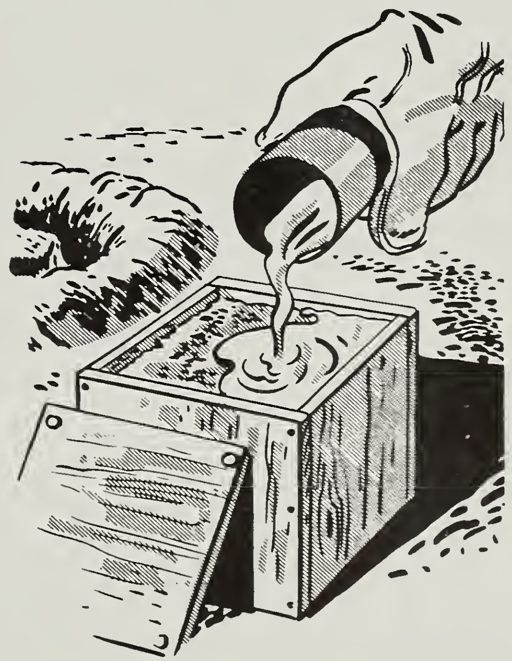
To obtain a box sample:

1. Excavate as for a chunk sample, then trim sample to size slightly smaller than box.



2. Remove top and bottom from box and place over sample.
3. Fill sides with paraffin, then pour paraffin over top of sample and replace bottom.

4. Cut off sample, remove box containing sample from hole, and turn right side up.



5. Trim surface of sample and seal with melted **paraffin**, then replace lid.

Figure 2-3. Box Samples

Bore Holes

General

Bore holes represent a common method of exploration in making sub-surface investigations. Various types of drilling equipment and tools are available for advancing bore holes. Bore holes are advanced for (1) general investigation during preliminary examination, (2) obtaining representative disturbed samples, (3) for advancing and cleaning holes to specific horizons for logging, sampling, or conducting tests, (4) for advancing holes to bedrock to delineate rock surfaces, and (5) for installing piezometers and relief wells, Only the more common methods of boring for SCS work are described in this section.

Hand-auger Borings

Hand augers are useful for advancing holes to shallow depths for preliminary examination of sites. They may be used to depths up to 20 feet. Beyond that depth they become cumbersome to handle and boring is slow. Usually a bucket-type hand auger is found to be most useful because it provides samples for inspection without the excessive mixing of materials which occurs when a helical or wormtype bit is used.

Several kinds of motorized hand augers (post-hole augers) are available on the market. Although a shallow hole can be advanced rapidly with this type of equipment, the limitations in use are similar to those of hand augers.

Power-auger Boring

Power augers mounted on trucks or jeeps are used for dry boring in unconsolidated materials. Bore holes are advanced by rotating a cutting bit into the materials. A wide variety of materials may be bored with power augers. They are not suitable for use in materials containing cobbles or gravel, hard or cemented soils, or saturated cohesionless soils. Unstable materials require casing particularly below the water table.

In power-auger boring, materials are brought to the surface by lifting the bit and removing materials contained therein, or, by use of a continuous flight or helical auger which spirals the material to the surface by mechanical means. Raising the bit for cleaning each time results in slow advancement of holes. However, it facilitates inspection of materials from strata of known depth. The continuous flight auger provides a more rapid method of advancing holes. The material brought to the surface is badly mixed by the action of the flight auger and correlation with depth is always questionable so that definite identification and location of changes of strata is not possible.

The use of power augers usually leaves the hole clean and dry above the water table and the materials in the bottom of the hole relatively undisturbed.

Percussion or Churn Drilling

Percussion or churn drilling is an inexpensive method of advancing bore holes through hard consolidated soils, rock, coarse gravel deposits, or formations containing boulders. It is often used in conjunction with auger boring.

The hole is drilled by the impact and cutting action of a heavy, chisel-edged drilling tool that is alternately raised and dropped by means of a cable attached to a walking arm, "jerkline," crank, or other means of raising and dropping the cable and tools.

A small amount of water in the bore hole permits the loosened rock and soil to be mixed into a slurry as the hole is advanced. When the carrying capacity of the slurry is reached, the drilling bit is removed from the hole and the slurry removed by means of a bailer or sand pump, usually operated by a separate winch and light cable or "sand line." When drilling clay materials, the addition of sand will augment the cutting action of the bit. Also, when drilling in cohesionless materials, the addition of clay will increase the carrying capacity of the slurry.

Except in extremely stable materials, churn-drill holes usually require casing. It is desirable that the hole be advanced ahead of the casing, but this is often impossible in soft or sandy soils. Because of mixing of materials, the materials removed from the hole are non-representative and inadequate for delineation of particular strata.

Wash Borings

A wash boring is a means of rapidly advancing a hole by a striking or rotating, cutting or chopping tool and by jetting with water which is pumped through the hollow drill rod and bit. The method usually requires use of casing. Cuttings are removed from the hole by the water circulating upward between drill rod and casing. The cutting tool is alternately raised and dropped by tightening and slacking of a line wrapped around a cathead. A tiller attached to the drill rod permits rotating the rod and cutting tool. The material brought to the surface in the circulating water is non-representative of materials in place. Consequently, positive identification of particular strata is not possible when holes are advanced by the wash-boring method.

Displacement Borings

This method consists of forcing a tube into soil materials and withdrawing material which is retained inside the tube. Tubes may be driven by use of a drive hammer or pushed using a jack or hydraulic cylinders against the weight of the rig. Displacement

boring can be made in clays. silts and relatively stable materials free from gravel, cobbles, and boulders. The sampler, when withdrawn, acts as a piston in the hole causing more excessive caving than other methods of boring. Although highly recommended for logging purposes, continuous drive boring represents a slow method of advancing holes when needed for purposes other than loggine. Even minor changes in soil materials can readily be detected by extruding samples from the tubes. However, when used for logging purposes, the hole should be advanced by other means, such as auger boring, and tubes smaller than the hole diameter should be used in order to provide wall clearance. Displacement boring for advancing holes is generally impractical for diameters larger than three inches.

Rotary Drilling

In rotary drilling, the bore hole is advanced by rapid mechanical rotation of the drilling bit which cuts, chips, and grinds the material at the bottom of the hole into small particles. The cuttings are normally removed by pumping water or drilling fluid, from a sump, down through the drill rods and bit and up through the hole, from which it flows into a settling pit and back to the sump. Compressed air is available on many newer rigs to remove the cuttings from the hole. However, this is not very satisfactory in wet formations which are frequently encountered in dam site investigations.

A reverse water circulation is employed on rigs used to drill large-diameter holes such as water wells. In this case the drilling fluid passes down through the hole and up through the drill rods. The higher upward velocity of the fluid through the drill rods facilitates removal of cuttings from large holes.

Holes can be advanced in a wide variety of materials, including sound rock, by rotary-drilling methods. Rotary drilling may be the only practical method of advancing holes and obtaining undisturbed core samples from certain types of soil and rock materials. Rotary-drilling equipment is versatile. Any of the foregoing methods of advancing holes can be used with rotary-drilling equipment.

Geophysical Methods

Geophysical methods may be used to supplement test holes for geologic exploration. It is desirable to have a limited number of test holes for interpretation of results obtained by geophysical procedures. Geophysical methods are rapid and economical and may reduce the number of test holes that are required at a particular site to establish geologic continuity. With test hole control, geophysical methods may be helpful in delineating the bedrock profile and determining the continuity of strata between borings for certain types of geologic conditions.

Seismic

The seismic refraction method is based on the variable rate of transmission of seismic or shock waves through materials of varying densities composing the earth's crust. The nature of material is inferred from the rate of transmission of sound. Typical rates of transmission for different types of materials are shown in Figure 2-4.

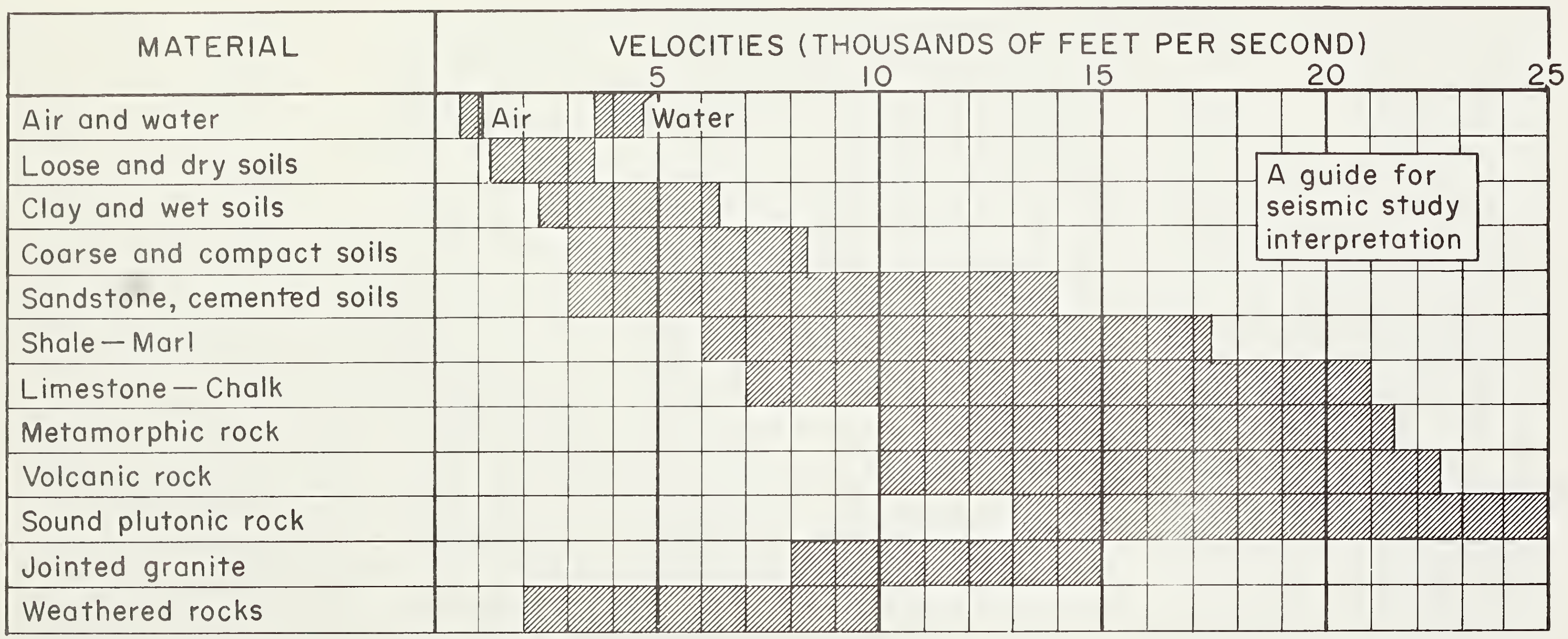


Figure 2-4. Velocities of Longitudinal or Compression Waves.

Hvorslev, M.J., ASCE, Nov. 1949

Several types of operating procedures have been developed. In one method, small explosive charges are set off in shallow holes. The seismic wave so generated is picked up and its time of arrival is recorded at several surface detecting points. The travel time of the wave to these recording points is measured and the wave velocity of different strata may be calculated. From these data the depths and probable character of various beds or layers can be inferred. See Figure 2-5 for schematic drawing.

Light-weight portable refraction seismographs have been developed which accurately measure the time interval for travel of a sound wave from a source to the instrument. In this method the shock wave is created by a sledge hammer blow on the ground and is picked up by a geophone. These portable units may be equipped with one or more geophones. The mechanical energy of the wave is transformed by the geophone into an electrical signal which in turn is fed into the receiver. The time interval of the seismic wave is read directly on the instrument by means of binary counters or is recorded on film or paper.

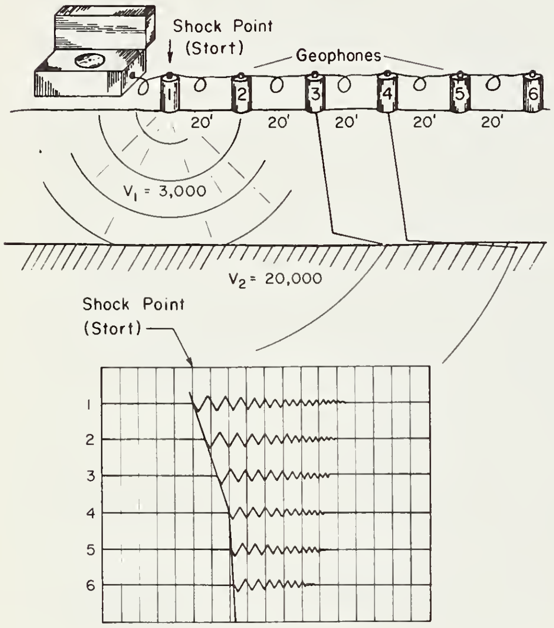
The velocity of the wave is then computed from the registered time and the distance of the hammer from the instrument. Changes in the physical characteristics of underlying materials are indicated by the changes in velocities (distance/time) recorded by the instrument.

The instrument, when the shock wave is created by a hammer blow, is limited to depths of about 50 feet and to rather simple problems of geology. For example: (1) a single discontinuity between two formations, (2) dipping discontinuity, and (3) two horizontal discontinuities, providing each formation becomes progressively denser with depth. Because the velocity of sound in water is about 5,000 fps., groundwater tables can be delineated in formations having seismic velocities less than that of water. The equipment is relatively inexpensive compared to seismographs used for oil and mineral exploration work. Although used only to a limited extent in foundation exploration, it appears to have possibilities for preliminary studies and for reducing the number of test holes needed to extrapolate continuity.

Electrical Resistivity

The resistivity, or electrical resistance, of earth materials can be determined readily by causing an electrical current to flow through the materials being tested. Usually, four electrodes are set in the ground in a line and at an equal distance apart. A set of batteries and a milliammeter are connected in series with the outer pair of electrodes. These are the two current electrodes. A potentiometer for measuring voltage is connected with the inner pair of electrodes. They are the potential electrodes. In many types of instruments a device, such as a commutator, is incorporated in the circuit

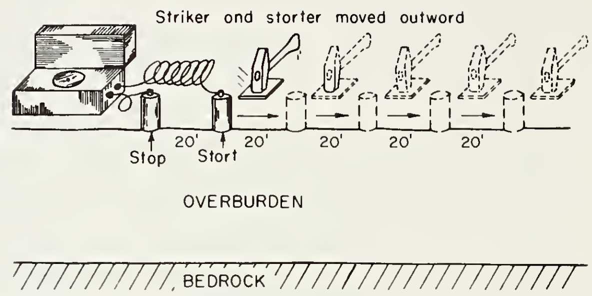
MULTI-TRACE RECORDING REFRACTION SEISMOGRAPH



SEISMOGRAM

Time lines 0.01 sec. apart

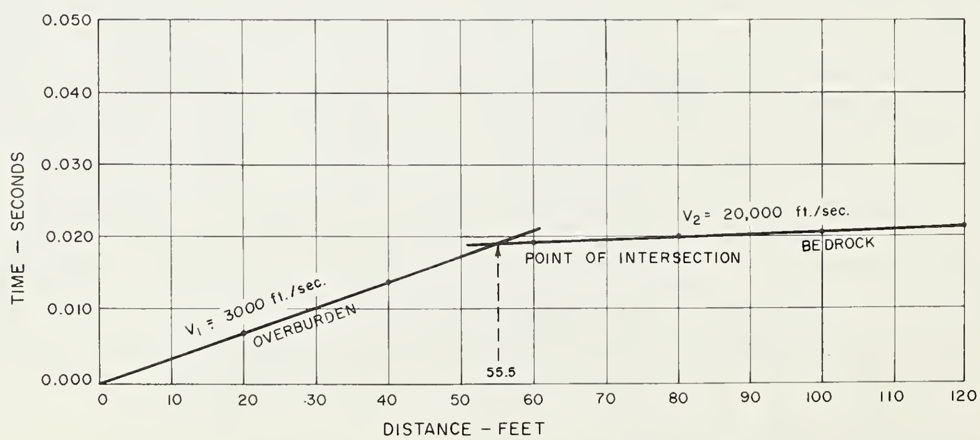
PORTABLE REFRACTION VISUAL TIMER



DISTANCE (FEET)	TIME (MILLISECONDS)
20	5, 6, 7, 6, 7, 7
40	13, 12, 14, 13, 13
60	19, 18, 19, 19
80	20, 21, 20, 20
100	21, 21, 21
120	22, 23, 46, 22, 22

TIME RECORD

Visual readings from timer



V = Shock wave velocity = $\frac{D}{T}$ = ft./sec.

D = Distance to point of intersection (feet).

T = Time to point of intersection (seconds).

H = Overburden thickness = $\frac{D}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$

V_1 = Overburden velocity

V_2 = Bedrock velocity

Figure 2-5. Seismic Refraction Methods

for the purpose of reversing the polarity of the current electrodes (Figure 2-6). This procedure permits the registering of several potential readings for comparison. By measuring the current and the potential drop between the two inner potential electrodes, the apparent resistivity of the soil to a depth approximately equal to the spacing interval of the electrodes can be computed. The resistivity unit may be designed so that the apparent resistivity is read directly on a potentiometer using the principle of a wheatstone bridge.

In sediments or loose rock the resistivity meter will show a marked drop in potential at the water table. However, in solid rock the greater resistance of the material will often mask the presence of the water table.

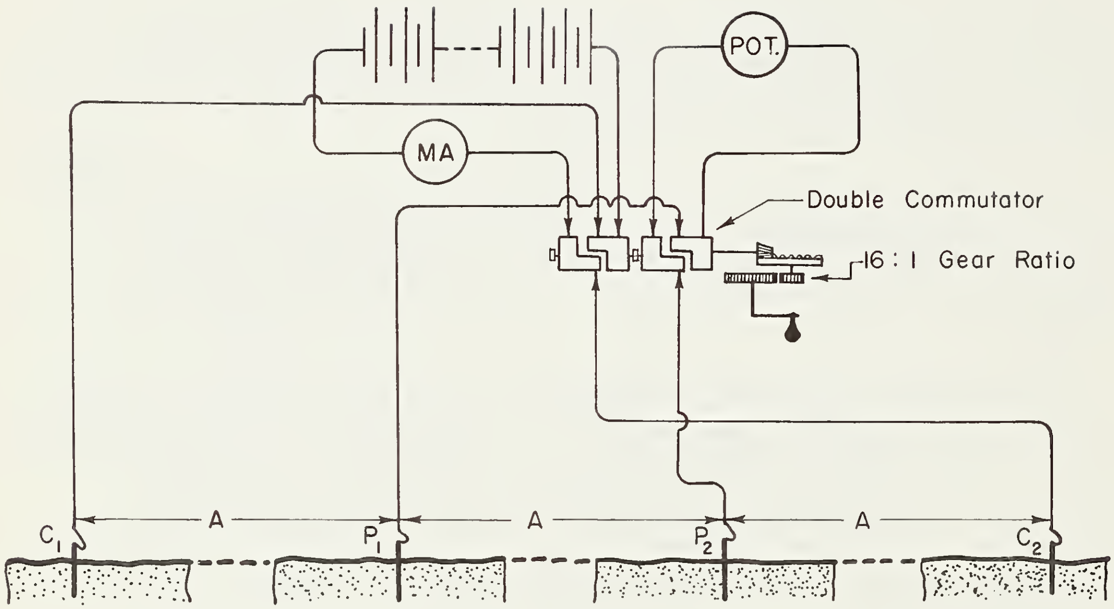
In "resistivity mapping" or "traverse profiling," the electrodes are moved from place to place without changing their spacing. The resistivity and any anomalies to a depth equal to the spacing of the electrodes can be determined for the various points.

In "resistivity sounding" or "depth profiling" the center point of the setup remains fixed while the spacing of the electrodes is changed. By plotting the apparent resistivity as a function of the electrode spacing, the subsurface conditions may be indicated. A break or change in curvature of the plotting will generally be noted when the electrode spacing equals the depth to a deposit with a resistivity differing from that of the overlying strata.

The electrical resistivity method and refraction seismographs have been used complementing each other with good results, particulary in delineating gravel lenses, bedrock and the groundwater table. However, salt water and saline soils have marked influence on conductivity. Excellent results have been obtained at swampy sites containing organic soils and saturated materials.

Field Penetration Test

This test provides a measure of the resistance of soil to penetration of the sampler. It also furnishes samples of the material penetrated for identification, classification, and other test purposes. This test is used to indicate relative in-place density of cohesionless and relative in-place consistency of cohesive foundation materials and for logging. Table 2-1 shows the relative density and consistence for various soils and blow counts.



Legend

MA – Milliammeter

POT. – Potentiometer

C_1, C_2 – Current Electrodes

P_1, P_2 – Potential Electrodes

A – Uniform spacing between electrodes

Figure 2-6. Diagram of Resistivity Apparatus

Table 2-1. Standard Penetration Resistance

Noncohesive Soils		Cohesive Soils	
Blows per Foot	Relative Density	Blows per Foot	Consistency
Less than 4	Very loose	Less than 2	Very soft
4-10	Loose	2-4	Soft
10-30	Medium	4-8	Medium
30-50	Dense	8-15	Stiff
Over 50	Very dense	15-30	Very stiff
		Over 30	Hard

Application

The standard penetration test is recommended for use in SCS work. It is most applicable to fine grained soils that are at or near saturation and to fairly clean, coarse grained sands and gravels at variable moisture contents. Materials below the water table may generally be considered to be saturated.

Equipment

Drilling equipment - Any equipment may be used that will provide a reasonably clean hole to insure that the test is performed on undisturbed material and that will drive and reclaim the sampler in accordance with the procedure outlined below. Where necessary, casing or hollow stem auger will be used to prevent caving. The hole will be at least 2¼ inches in diameter.

A, B, or N rod may be used, however A or B is preferred. If N rod is used, the minimum hole diameter should be 2 3/4 inches.

Bottom discharge fishtail bits, jetting through open tube or sand or water bailers will not be used to advance holes.

Split-tube sampler - The sampler shall have an outside diameter of 2 inches. It shall consist of (1) a hardened steel driving shoe at least 3 inches in length with an inside diameter at the cutting head of 1 3/8 inches. It shall be sharpened by tapering the last 3/4 inch to a cutting edge not greater than 1/16 inch thick. Dented, distorted, or broken shoes shall not be used; (2) the split tube shall have a minimum length of 16 inches and an inside diameter of 1 3/8 <u>or</u> 1 1/2 inches; (3) the coupling head shall have a minimum length of 6 inches. It will have four vents each with a minimum diameter of 1/2 inch <u>or</u> it shall contain a ball check valve and no side vents. (Figure 2-7)

Hammer - The drive hammer shall weigh 140 pounds and have a 30-inch stroke (free fall). Any type of hammer may be used as long as there is no interference with its free fall and its energy is not reduced by friction on the drill rod, guides, or other parts of the equipment.

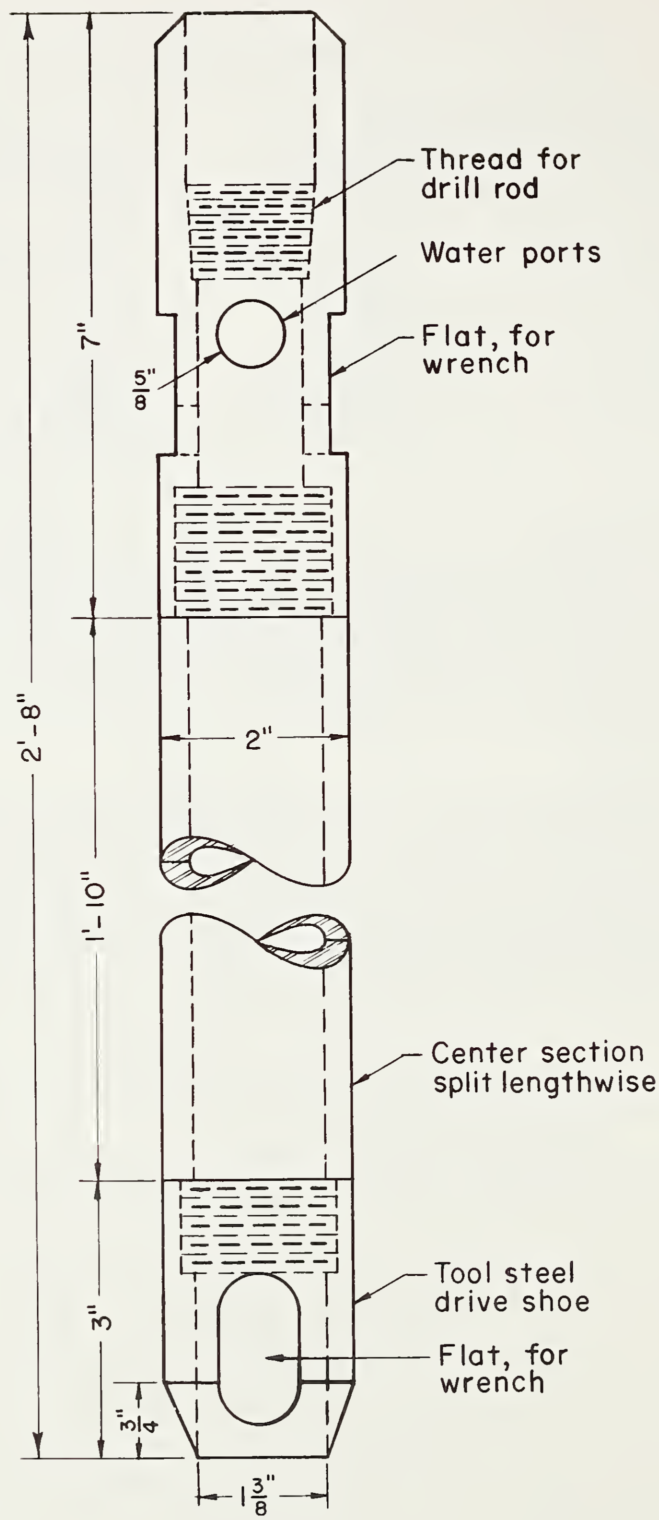


Figure 2-7. Split-Barrel Samples

Procedure

Cleaning hole - Clean the hole to the sampling elevation by use of equipment that will not disturb the material to be sampled. Do not use bottom discharge fishtail bits, jetting through an open tube, or sand water bailers. Take samples at each change in stratum and at intervals not greater than 5 feet. Never drive casing (or hollow stem auger) below the depth to which the hole is to be cleaned out.

The standard penetration test - Lower the split-tube sampler to the bottom of the cleaned hole. With the water level in the hole at the groundwater level or above, drive the sampler 6 inches with light blows so it will not be overdriven. Then drive the split-tube sampler 12 inches or to refusal, by dropping the 140-pound hammer 30 inches and record separately the number of blows required for each 6 inches of this 12-inch penetration test drive. Penetration of less than 1 foot in 100 blows is generally considered refusal. The blow count is the total number of blows required to drive this last 1 foot, or, with refusal, the number of inches penetrated by 100 blows.

The split-tube sampler is not to be used as a chopping bit. Where a boulder is encountered, it should be penetrated by other means (drilled or by chopping bit) or bypassed.

Remove the sampler from the hole and open it. Identify and classify the material or materials, record the percent recovery, place typical sample or samples in jars (without jamming or compressing), seal jars with wax and label. Label to show all data as to site location, location of hole and depth represented by sample, field classification, blow count, and percent recovery.

Vane Shear

The vane shear test provides a field method for determining the shearing resistance of a soil in place. See Figure 2-8. The vane, attached to the end of a rod, is forced into an undisturbed soil to be tested and rotated at a constant rate by means of a torque wrench or other calibrated torsion device attached to the rod. The moment or torque required to turn the vane is an indication of the shear strength of cohesive soils. Vane shear testing should be closely coordinated and carried out under the direction of the Unit Geologist.

Permeability Investigations

The coefficient of permeability is the rate of discharge of water under laminar flow condition through a unit cross-sectional area of a porus medium under a unit hydraulic gradient and standard temperature conditions. There are two temperatures which are used as standard. These are 60^{O}F and 20^{O}C. Two

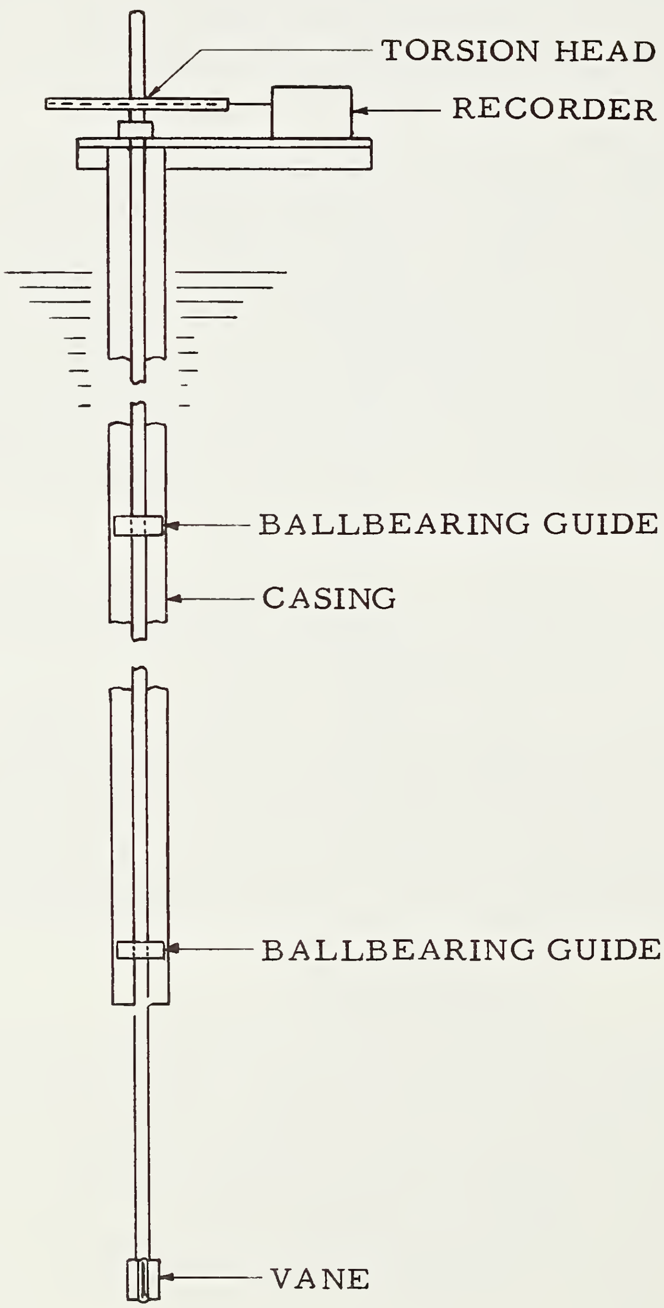


Figure 2-8. Idealized Vane Shear Apparatus

commonly used units for expressing the coefficient of permeability are: gallons per square foot per day under a hydraulic gradient of one foot per foot, and cubic feet per square foot per day under a hydraulic gradient of one foot per foot. This latter unit is commonly expressed as feet per day and treated as a velocity. This would be the discharge velocity under the conditions of unit head and standard temperature. Dividing this value by the porosity of the material will give the average velocity or dividing by the effective porosity of the material will give what Tolman[1/] terms the effective velocity or the actual velocity of the moving water.

Various field tests are used to determine water loss in rock formations. The test is carried out by means of sealing off portions of bore holes, introducing water under pressure, and measuring rate of water loss into the formation. Interpretation of results of pressure tests are illustrated in Figure 2-9. Pressure testing permits delineation of zones of leakage, for estimating grouting requirements or other treatment which may be needed to reduce water movement. Where pressure testing is required, bore holes should be tested in intervals of five feet or less.

Pressure-testing equipment - The apparatus commonly used for pressure testing foundations in rock consists of expansion plugs or packers set five feet apart, which may be expanded to seal off sections of a drill hole. See Figure 2-10. Water lines are so arranged that water may be admitted either below the bottom expansion joint or from a perforated pipe between the two expansion joints. The water lines are connected through a pressure relief valve, pressure gage, and water meter, to a pressure pump.

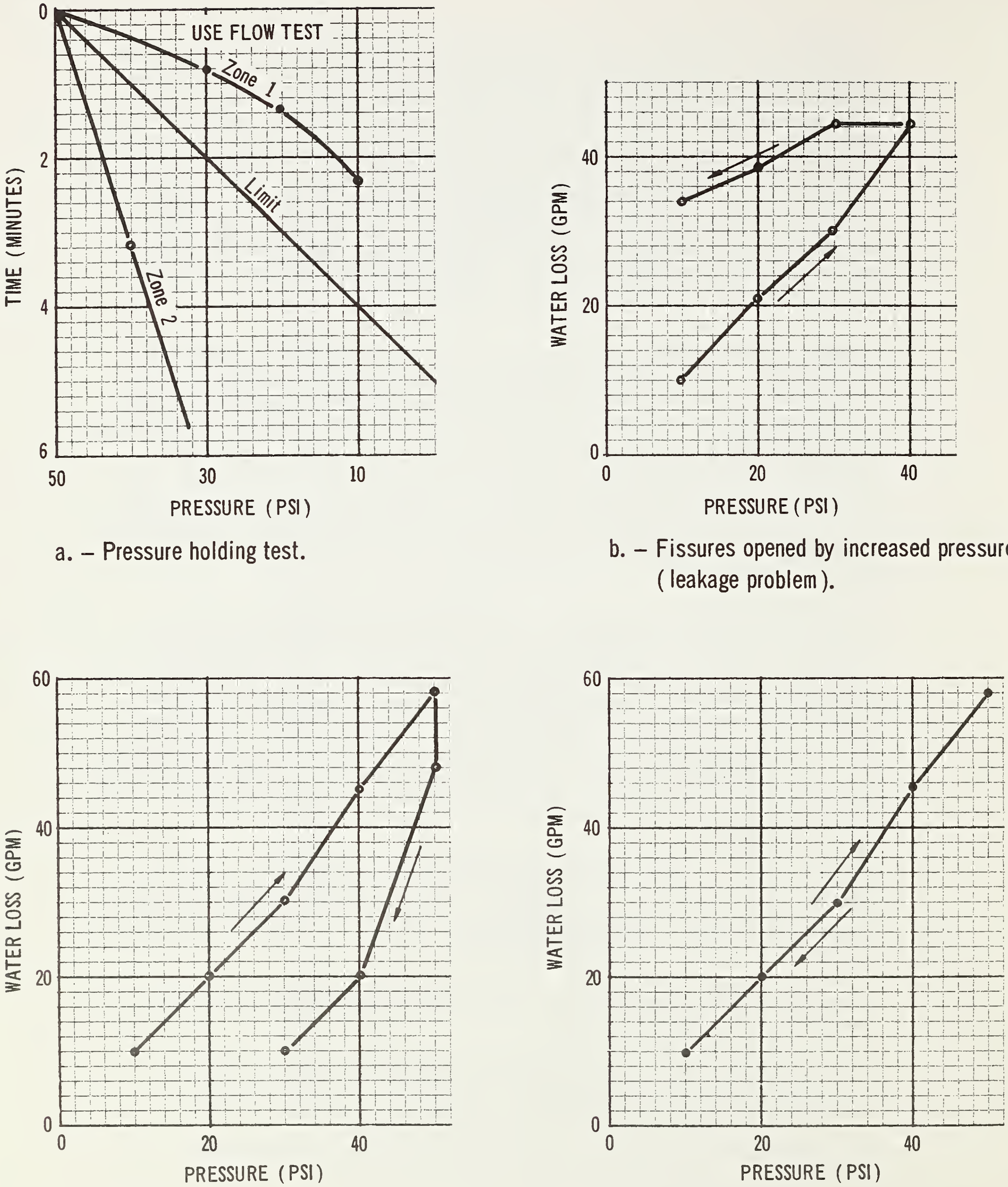
Water pumps having a minimum capacity of 50 gpm at discharge pressures of 100 psi are needed. Additional equipment includes accessory valves, gages, stopcocks, plugs, and tools necessary for maintaining uninterrupted tests.

Pressure-testing procedure - The following procedures apply in conduction a pressure test:

1. Lower the packer assembly in the bore hole to the predetermined depth of testing.

2. Expand the packers to seal off bore hole in section to be tested.

1/ Tolman, C. F., Ground Water, McGraw-Hill Book Co., Inc., New York and London, 1937, p. 216.



a. – Pressure holding test.

b. – Fissures opened by increased pressure (leakage problem).

c. – Self–sealing formation (no leakage problem).

d. – Stable condition (leakage problem).

Figure 2-9 Sample Plots of Pressure-Test Data

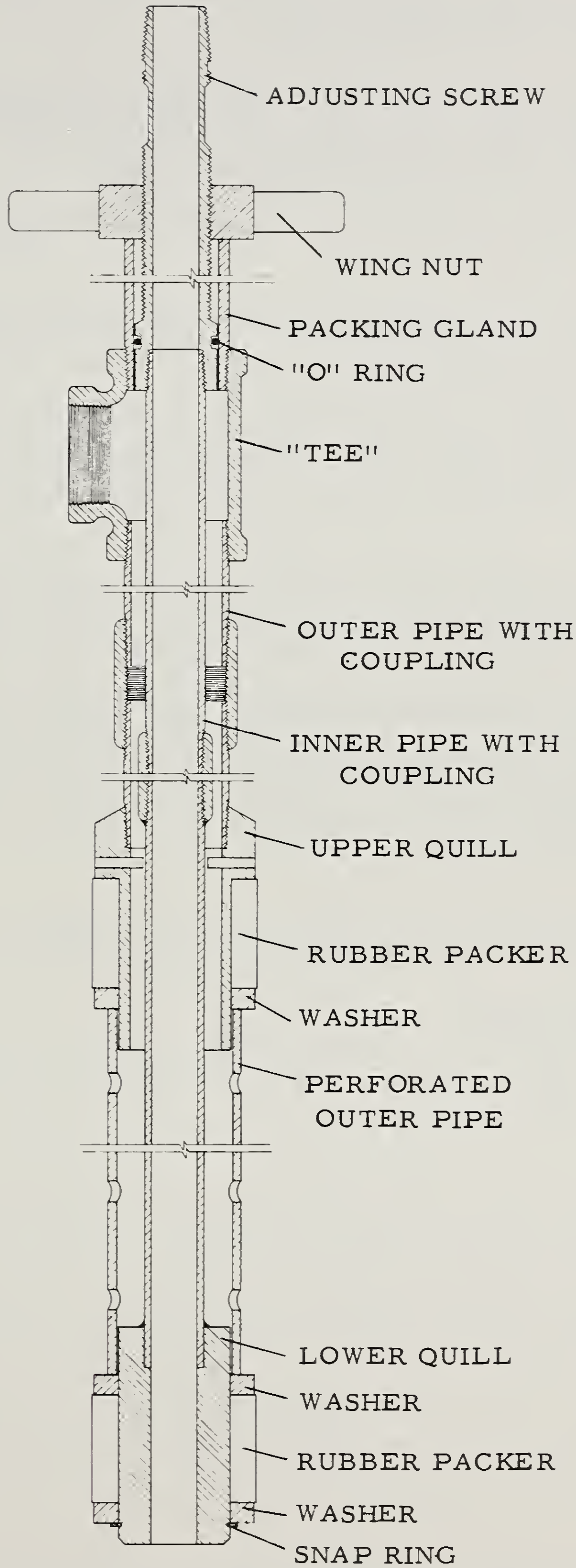


Figure 2-10. Pressure-testing Tool

3. Water under pressure is forced into the bore hole between packers by the pressure pump.

4. Adjust water pressure. Maximum water pressure at the upper packer for the holding test should not exceed one psi (pounds per square inch) per foot of hole depth.

5. When the initial test pressure is reached, close stopcocks and cut off pressure pump.

6. Record the pressure drop for various intervals of time to determine rate of drop.

7. If pressure drop exceeds 10 psi per minute, continue investigation with a flow test. A pressure drop of less than 10 psi per minute is evidence that no appreciable leakage occurs in the zone tested.

8. Where additional testing is indicated a packer type flow test may be used (see pumping-in tests below). In some types of material it may be desirable to determine rates of flow at several different pressures in ascending order from the lower to the higher pressure. The maximum pressure should not exceed 0.43 times the vertical distance in feet between the test elevation and the elevation of the emergency spillway. Then recheck these rates in descending order from the higher to the lower pressure. See Figure 2-9, b, c, and d.

9. Release water pressure between packers, contract expanders and move unit to next interval to be tested.

Pumping-in Tests
Each of the following tests applies to a certain set of conditions. If all of these conditions are not present, erroneous permeability coefficients will result.

Open-end tests - Open-end tests can be made in permeable formations either above or below the water table and with the use of gravity flow or pressure flow. The stratum being tested should have a thickness at least ten times the diameter of the test well. The test is based on the amount of water accepted at a given head by the formation through the bottom of a pipe or casing. Clear water must be used if valid results are to be obtained. It is also desirable that the temperature of the water being added be higher than the temperature of the groundwater to prevent the formation of air bubbles in the formation.

The casing should be sunk to the disired depth, leaving a foot or two protruding above ground. It is then carefully cleaned out just to the bottom of the casing. Drilling muds should not

be used in making holes. Cleaning should proceed, using clear water, until clear water is returned to the surface. A standard cleanout auger (figure 2-24) or other tool with jet deflector or low-pressure jet should be used to avoid disturbance of the material below the casing. Below the water table the hole should be kept full of water at all times during cleaning to avoid forcing of the materials up into the casing by water pressure from below. This makes it necessary to determine the normal water level in the hole before cleaning.

The test is then begun by adding clear water into the hole, maintaining a constant water level in the casing, until a steady rate of intake is established. If pressure is applied, water should be pumped until rate of inflow and pressure remain steady. Above the water table a constant level and intake rate are rarely attained and a slight surging of the water level or pressure at constant inflow may occur. When the oscillations become regular for a few minutes the test can proceed. An anti-surge device consisting of a capped, air-filled, stand pipe may be placed in the supply line near the pressure gage. This will dampen the surges and make gage readings easier to take.

The length of the test should be measured with a stop watch. Normally, ten minutes should be long enough. The volume of water should be measured with a water meter or other method accurate within 1 or 2 percent. The rate of flow (Q) is then computed by dividing the volume by time. Q is usually recorded as gallons per minute.

Above the water table, head (h) is measured from the bottom of the hole to the elevation of the maintained water level. Below the water table it is measured from the groundwater level to the maintained level. If pressure is applied, head is measured from the bottom of the hole or the normal water level to the elevation of the gage, plus the applied pressure. If the gage reads in pounds per square inch, the pressure reading is multiplied by 2.31 to convert it to feet of head (1 psi = 2.31 ft.).

The size of casing is usually measured in inches. For this test, the radius (r) is the inside radius of the bottom of the casing.

Figure 2-11 illustrates the conditions and procedures discussed above.

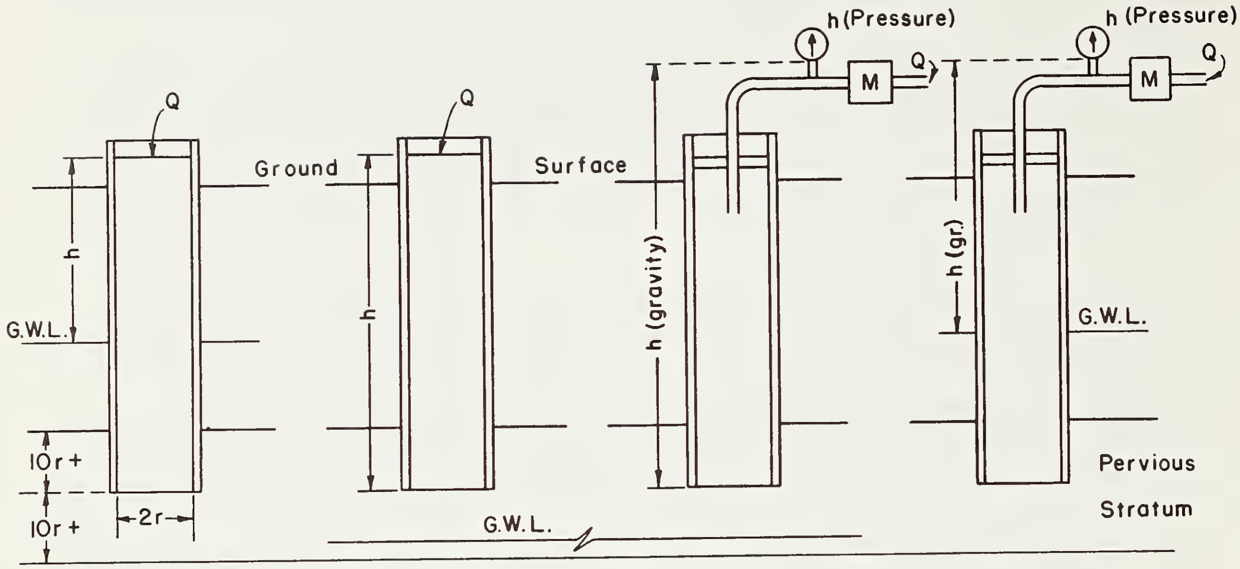


Figure 2-11 Open-end Permeability Test

Electric analog experiments conducted by the U.S.B.R. gave the following relationship between the above data and permeability:

$$k = \frac{Q}{5.5rh} \tag{1}$$

where,

k = coefficient of permeability,

Q = constant rate of flow into the hole,

r = internal radius of the bottom of the casing,

h = differential head of water

Any consistent set of units can be used. As an example:

Data obtained in Field	Conversion to Consistent Set of Units
Q = 8.3 gallons per minute	= 1598 cubic feet per day
r = 1.5 inches	= 0.125 feet
h = 7.3 feet	= 7.3 feet

$$k = \frac{1598 \text{ ft}^3/\text{day}}{5.5 \times 0.125 \text{ ft} \times 7.3 \text{ ft}} = 318 \text{ ft}^3/\text{ft}^2/\text{day}$$

For convenience, the formula can be written:

$$k = \frac{420Q}{rh} \tag{2}$$

where,

k is in cubic feet per square foot per day,

Q is in gallons per minute,

r is in inches, and

h is in feet.

Using the same data as in the above example:

$$k = \frac{420 \times 8.3}{1.5 \times 7.3} = \frac{3486}{10.95} = 318 \text{ ft}^3/\text{ft}^2/\text{day}$$

It should be remembered that this test is an approximation and should not be considered to give a precise value for permeability. It has the advantage of being a simple test which can be performed during normal drilling operations. It gives a good indication of relative permeabilities at various depths. The test should not be performed with the bottom of the hole less than a distance of 10r from either the top or bottom of the strata being tested.

Packer tests

Packer tests are commonly used for flow testing of bedrock formations after pressure holding tests have indicated a need for permeability determinations. In bedrock they are usually conducted with the use of packers, in which case different sections of a completed hole can be tested by moving the packers. They can be conducted in unconsolidated materials between the bottom of the hole and the end of the casing or packer set in the bottom of the casing. In unconsolidated material, it must be certain that there is no space between the outside of the casing and the wall of the hole. If the hole is too large, water will escape upward outside the casing, giving erroneous results. Driving the casing a few inches beyond the bottom of the bored hole and cleaning it out will alleviate this.

If unconsolidated materials cave into the hole, the test can be performed in the following way: (1) Drive the casing to the bottom of the hole and clean it out. (2) Accurately measure the depth of the hole. (3) Pour a measured volume (V) of coarse sand or gravel into the casing, filling it to a depth slightly in excess of the length to be tested. The permeability of the added

gravel must exceed the permeability of the strata being tested by at least the ratio that the cylindrical and end area of the test section exceeds the end area of the casing. Otherwise, grossly erroneous rates will result. This determination must be based on judgment, obviously, because the permeability of the formation is unknown. If the length of the test section is kept short, using 3/8 to 1/4 inch gravel will usually be adequate for testing unsorted sands and gravels. (4) Withdraw the casing to the top of the test section. Be careful not to pull the casing above the top of the gravel inside the hole. (5) Accurately measure the depth to the gravel pack. Subtracting this depth from the depth of the bottom of the hole is the factor L (see Figure 2-12), needed in computations. (6) Determine the mean radius (r) of the test section by the following formula:

$$r \text{ (ft)} = \sqrt{\frac{V \text{ (cu ft)}}{\pi L \text{ (ft)}}} \tag{3}$$

where,

r = radius of test section, (ft.),

V = volume of gravel added to hole, (cu. ft.),

L = length of test section, (ft.).

After completion of the test, if desirable, the hole can be deepened and a test run at a lower elevation.

In tests between two packers, used in rock where the hole will stand, it is usually desirable to complete the hole to final depth, clean it out, fill it with water, and start testing at the desired intervals from the bottom upward. In this way the entire hole can be tested without removing the apparatus from the hole.

In tests below the watertable, head (h) is measured in the same way as for the open-end test. That is, the vertical distance, in feet, from the watertable to the pressure gage, plus 2.31 times the gage pressure reading. Above the watertable it is measured from the mid-point of the test section to the pressure gage plus the applied pressure in feet of water. See Figure 2-12.

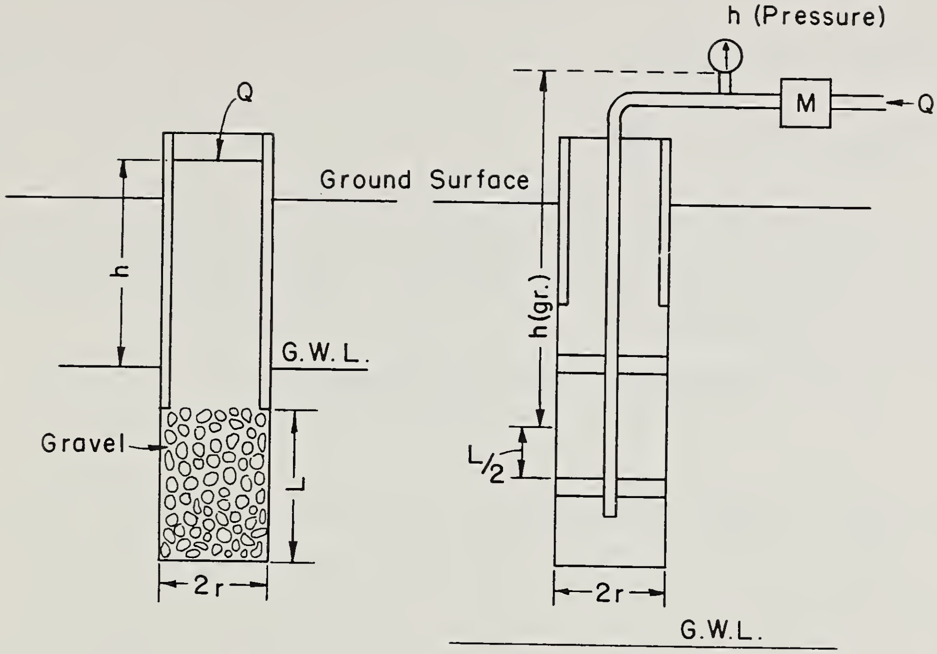


Figure 2-12 Packer-type Permeability Test

Where the length of the test section (L) is equal to or more than 5 times the diameter of the hole ($L \geqq 10r$), the formula used to compute permeability is:

$$k = \frac{Q}{2\pi Lh} \log_e \left(\frac{L}{r}\right) \tag{4}$$

where,

k = permeability,

Q = constant rate of flow into the hole,

L = length of the test section,

h = differential head of water,

r = radius of test section

If the length of section being tested is less than five times the diameter, the relation is best described by changing the natural logarithm ($\log_e$) or $\frac{L}{r}$ in the above formula to the arc hyperbolic sine ($\sinh^{-1}$) of $\frac{L}{2r}$:

$$k = \frac{Q}{2\pi Lh} \sinh^{-1} \left(\frac{L}{2r}\right) \tag{5}$$

Again, any consistent set of units can be used. If however, k is in cubic feet per square foot per day, Q in gallons per minute, and L, h, and r in feet, the formulas can be rewritten:

$$k = \frac{30.6Q}{Lh} \log_e \left(\frac{L}{r}\right) \qquad (6)$$

$$k = \frac{30.6Q}{Lh} \sinh^{-1} \left(\frac{L}{2r}\right) \qquad (7)$$

Table 2-2 below gives rounded values for the arc hyperbolic sines of numbers between 0.5 and 4.9. Table 2-3 gives rounded natural logarithms of numbers between 10 and 99. Other values can be obtained by interpolation.

Table 2-2 Arc Hyperbolic Sines of Numbers from 0.5 to 4.9

	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
0						.482	.569	.653	.733	.809
1	.88	.95	1.02	1.08	1.14	1.19	1.25	1.30	1.35	1.40
2	1.44	1.49	1.53	1.57	1.61	1.65	1.68	1.72	1.75	1.78
3	1.82	1.85	1.88	1.91	1.94	1.97	1.99	2.02	2.05	2.07
4	2.09	2.12	1.14	2.17	2.19	2.21	2.23	2.25	2.27	2.29

Table 2-3 Natural Logarithms of Numbers from 10 to 99

	+0	+1	+2	+3	+4	+5	+6	+7	+8	+9
10	2.30	2.39	2.48	2.56	2.64	2.71	2.77	2.83	2.88	2.94
20	2.99	3.04	3.08	3.13	3.17	3.21	3.26	3.29	3.33	3.36
30	3.39	3.43	3.46	3.49	3.52	3.55	3.58	3.61	3.64	3.66
40	3.69	3.71	3.74	3.76	3.78	3.80	3.83	3.85	3.87	3.89
50	3.91	3.93	3.95	3.97	3.99	4.00	4.02	4.04	4.06	4.07
60	4.09	4.10	4.12	4.14	4.16	4.17	4.19	4.20	4.22	4.23
70	4.25	4.26	4.27	4.29	4.30	4.31	4.33	4.34	4.35	4.37
80	4.38	4.39	4.40	4.42	4.43	4.44	4.45	4.46	4.47	4.49
90	4.50	4.51	4.52	4.53	4.54	4.55	4.56	4.57	4.58	4.59

Following is an example of the use of the above formulas and tables:

Given:

$r = 0.125$ ft

$L = 2.1$ ft $\quad L > 10r$, use formula (6)

$h = h$ (gravity) + h (pressure)

h (gravity) = distance from ground water level or mid-point between packers to gage

= 3.7 feet

h (pressure) = 6 p.s.i. (gage reading)

= 13.9 feet (gage reading X 2.31)

= 3.7 + 13.9 = 17.6 feet

$Q = 3.6$ g.p.m.

$$k = \frac{30.6Q}{Lh} \log_e \left(\frac{L}{r}\right) \qquad (6)$$

$$L/r = \frac{2.1 \text{ ft}}{.125 \text{ ft}} = 16.8; \ \log_e 16.8 = 2.81 \text{ (from table 2-3)}$$

$$k = \frac{30.6 \times 3.6 \times 2.81}{2.1 \times 17.6} = 8.35 \text{ ft per day}$$

Well permeameter method - The open-end test and packer test described above are most practical in fairly permeable materials. That is, where the coefficient of permeability is one foot per day or greater. The well permeameter is best suited for low permeability materials. It is often useful in resevoir bottoms and canals to determine leakage potentials.

Since, in this test, the rate of inflow (Q) is usually very low, flow meters cannot be used and the volume of water used must be measured by some other method. An open-ended drum, calibrated in 1-gallon increments, is a convenient device. Also, since the test is of long duration and inflow rates are low, the water level must be maintained by use of a float valve. Any standard bob-float stock-watering valve with sufficient capacity to maintain the water level and with a counterbalanced operating arm can be used. The counter-balance allows the float to be suspended from the operating arm by means of a chain which can be lowered into the hole. The elevation of the water surface in

the hole is controlled by the length of the chain. Figure 2-13 is an illustration of the test apparatus as it should be set up.

The hole for this test can be made by any convenient method, taking care that all compacted soil is removed from the side of the hole and that the bottom of the hole is clean. The hole can be of any desired dimensions as long as it conforms to the general rule that its depth should be between 10 and 150 times the radius.

After the hole is completed and cleaned, it is backfilled with a measured volume of clean, uniform, coarse sand or gravel to a level about 6 inches below the water level to be maintained. The sand serves the purpose of supporting the hole during saturation and is a means of determining the mean radius of the hole. The radius is computed as described under the section on packer tests. A short piece of galvanized casing should be placed above the sand as protection for the float. This casing should be smaller than the hole to allow water to move freely outside the casing to the elevation of the water level in the well. It can be held in place by pouring pervious sand between the outside of the casing and the well.

The water used in the test should be from the same source as the water which will permeate the strata after construction, if possible. This is because in some soils and waters, a base exchange reaction takes place which might increase or decrease permeability. If this does occur, using water from a common source will assure any change will be in the right direction. The water must be completely free from sediment. Its temperature should be slightly higher than the temperature of the soil or ground water to prevent the formation of air bubbles. Because of the wide range of temperatures at shallow depths, the results of this test must be corrected to a standard temperature. This may be either 20° C or 60° F. Therefore, the temperature of the water in the hole must be taken. If the test is of long duration, the temperature should be taken several times and averaged to make the correction. Table 2-4 lists the factors by which the results of the test must be multiplied to make the temperature corrections (C_t). The factors are derived by dividing the visosity of water at the given temperature by the viscosity of water at standard temperature (20° C and 60° F).

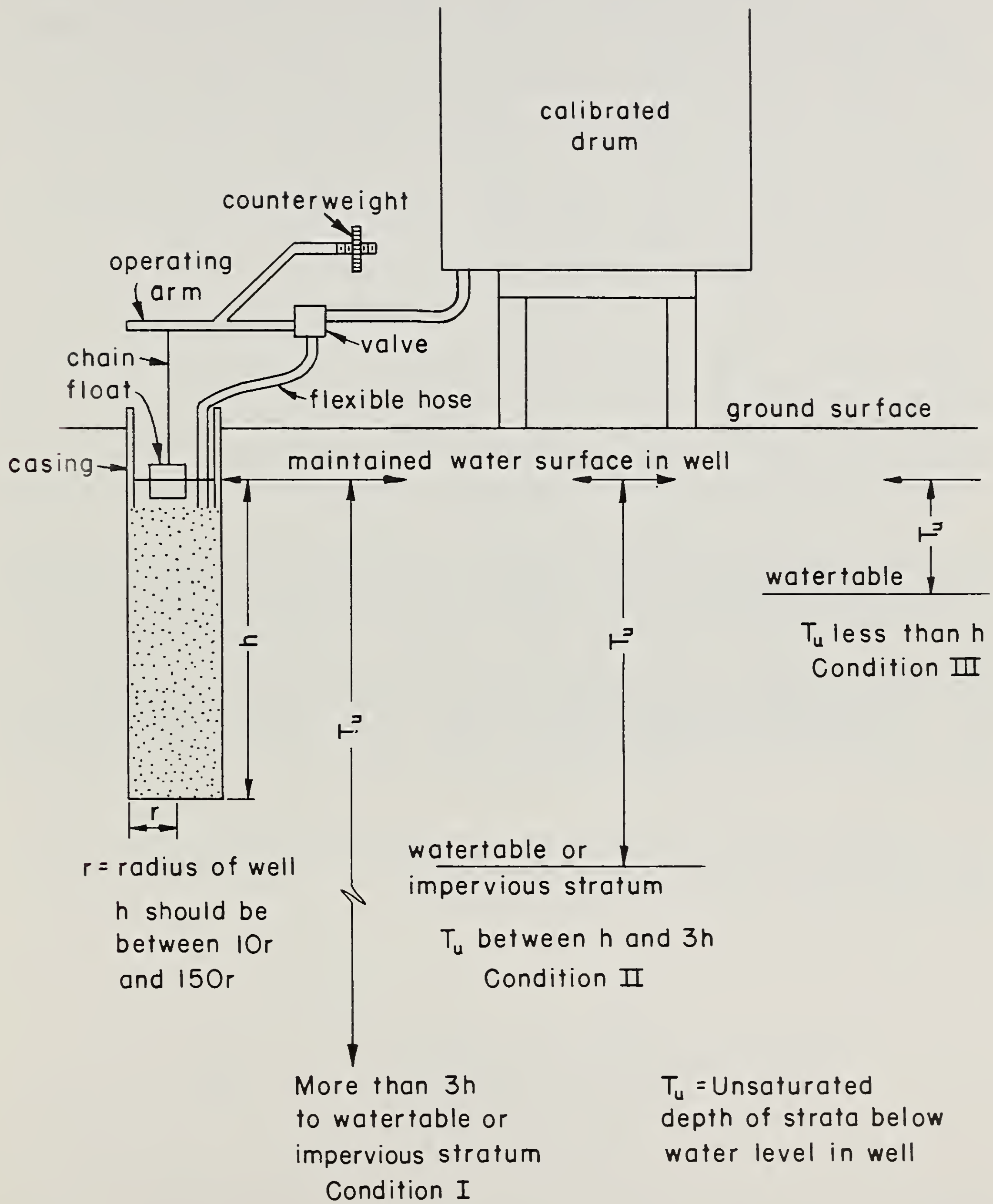


Figure 2-13 Well-permeameter Test

Table 2-4 Temperature Correction Factors

Water Temp. Degrees C	F	Correction Factor (C_t) to 20°C	60°F	Water Temp. Degrees C	F	Correction Factor (C_t) to 20°C	60°F	Water Temp. Degrees C	F	Correction Factor (C_t) to 20°C	60°F
10	50.0	1.30	1.17	17	62.6	1.08	0.97	24	75.2	0.91	0.82
11	51.8	1.27	1.14	18	64.4	1.05	0.94	25	77.0	0.89	0.80
12	53.6	1.24	1.11	19	66.2	1.02	0.92	26	78.8	0.87	0.78
13	55.4	1.20	1.08	20	68.0	1.00	0.90	27	80.6	0.85	0.76
14	57.2	1.17	1.05	21	69.8	0.98	0.86	28	82.4	0.83	0.75
15	59.0	1.13	1.02	22	71.6	0.95	0.85	29	84.2	0.81	0.73
16	60.8	1.10	0.99	23	73.4	0.93	0.84	30	86.0	0.80	0.71

The test should be run long enough to develop a saturated envelope in the soil around the well, but not long enough to build up the water table. In the more permeable materials the test should be run until a graph of time plotted against accumulative discharge for several hours is a straight line, indicating that a steady rate of discharge has been established. The straight portion of the curve should then be used for determining Q to compute permeability.

If a steady rate of discharge has not been established after approximately 8 hours, the minimum volume to be discharged can be determined from the following formula:

$$V_{min.} = 2.09\, Y_s \left[h \sqrt{\frac{2}{\sinh^{-1}\left(\frac{h}{r}\right) - 1}} \right]^3 \qquad (8)$$

where,

V_{min} = minimum volume to be discharged

Y_s = specific yield of soil being tested

h = height of water in well

r = radius of well.

The formula requires that the specific yield (Y_s) of the strata being tested be known. Specific yield is the amount of water which will drain from a soil by gravity flow. It is written as a decimal fraction of the saturated volume of the soil. For common soils, specific yields vary from 0.10 for fine grained soils to 0.35 for coarse grained soils. When the specific yield of the soil is not known, 0.35 should be used to give a conservative value for minimum volume. The test should be discontinued when the minimum volume has been discharged. Minimum volume can be determined from table 2-5, when h and r are known and specific yield is assumed to be 0.35. If the specific yield of the soil is known, the minimum volume determined from table 2-5, should be multiplied by the fraction $\frac{Y_s}{0.35}$ where Y_s is the known specific yield of the soil.

The field data needed to compute permeability are: (1) The rate of flow into the well in gallons per minute. (2) The mean radius of the well, in feet. (3) The height of the column of water in the well, measured from the bottom of the hole to the maintained water level, in feet. (4) The depth to the water table, if it is shallow, or the depth to an impervious layer or the water table (whichever is higher) if the water table is deep. (5) The temperature of the water in the well. The soil temperature should be determined also, to be sure that the water being used is warmer than the soil.

As illustrated in Figure 2-13, there are three different conditions which normally exist in the field. Each requires a slightly different formula for computing permeability.

Table 2-5 Minimum Volume, in Gallons, to be Discharged in Well Permeameter Where $Y_s = 0.35$

Radius of Well (r)		Height of Water in Well (h) Feet										
Inches	Feet	2	3	4	5	6	7	8	9	10	11	12
1.00	.083	25	70	145	265	410	620	900	1230	1620	2090	2660
1.25	.104	30	80	165	290	450	670	970	1310	1720	2220	2820
1.50	.125	35	85	180	310	480	720	1040	1390	1820	2350	2980
1.75	.146	35	95	195	335	510	770	1110	1470	1920	2480	3140
2.00	.167	40	100	210	355	550	820	1180	1550	2020	2600	3300
2.25	.187	40	110	220	380	580	870	1240	1630	2120	2720	3460
2.50	.208	--	115	235	400	610	910	1310	1710	2220	2840	3620
2.75	.229	--	125	250	420	640	960	1370	1780	2320	2960	3780
3.00	.250	--	130	260	440	670	1000	1430	1860	2410	3080	3940
3.25	.271	--	140	275	460	700	1050	1480	1930	2510	3200	
3.50	.292	--	145	290	480	730	1090	1540	2010	2610	3320	
3.75	.312	--	---	300	500	760	1130	1590	2080	2710	3440	
4.00	.333	--	---	315	520	790	1170	1640	2160	2800	3560	
4.25	.354	--	---	325	540	820	1210	1690	2230	2890	3680	
4.50	.375	--	---	340	560	850	1250	1740	2310	2980	3800	
4.75	.396	--	---	350	580	880	1290	1790	2380	3070		
5.00	.417	--	---		600	910	1330	1840	2450	3160		

Condition I exists when the distance from the water surface in the well to the water table or an impervious layer is greater than three times the height of water in the well. For this condition, equation (9) below, is used.

Condition II exists when the water table is below the bottom of the well, but the depth to the water table or an impervious layer is less than three times the height of water in the well. For this condition, equation (10) below, is used.

Condition III exists when the water table is above the bottom of the well. Equation (11) is used in this case.

Equation for condition I:

$$k = \frac{192\left[\sinh^{-1}\left(\frac{h}{r}\right)-1\right]\frac{Q}{2\pi}}{h^{2}}C_t \qquad (9)$$

Equation for condition II

$$k = \frac{192\log_e\left(\frac{h}{r}\right)\frac{Q}{2\pi}}{h^{2}\left(\frac{1}{6}+\frac{T_u}{3h}\right)}C_t \qquad (10)$$

Equation for condition III

$$k = \frac{192\log_e\left(\frac{h}{r}\right)\frac{Q}{2\pi}}{h^{2}\left(\frac{T_u}{h}-\frac{T_u^2}{2h^2}\right)}C_t \qquad (11)$$

where,

k = coefficient of permeability, cubic feet per square foot per day,

h = height of water in well, feet, measured from the bottom of well to maintain water level,

r = radius of well, feet,

Q = constant rate of flow into hole, gallons per minute,

T_u = unsaturated thickness between water level in the well and the water table or impervious layer,

C_t = correction to standard temperature.

Following are examples of each of the three conditions using the appropriate equation:

Condition I, where T_u is greater than 3h.

Given:

h = 5 feet

r = 0.125 feet

Q = 0.10 gallons per minute

T_u = 30 feet (greater than 3h; use equation 9)

T = temperature of water in well = 18° C

$$k = \frac{192\left[\sinh^{-1}\left(\frac{h}{r}\right)-1\right]\frac{Q}{2\pi}}{h^2} C_t \qquad (9)$$

$$\frac{h}{r} = \frac{5\text{ ft}}{0.125\text{ ft}} = 40;\ \sinh^{-1} 40 = 4.37$$

$$k = \frac{192 \times (4.37-1)}{25} \times 1.05 = 0.43\ ft^3/ft^2/day$$

The arc hyperbolic sines for $\frac{h}{r}$ from 10 to 200 can be obtained from table 2-6.

Table 2-6 Arc Hyperbolic Sines

	+0	+1	+2	+3	+4	+5	+6	+7	+8	+9
10	3.00	3.10	3.18	3.26	3.34	3.41	3.47	3.53	3.59	3.64
20	3.69	3.74	3.79	3.83	3.88	3.92	3.96	3.99	4.03	4.07
30	4.10	4.13	4.16	4.19	4.22	4.25	4.28	4.30	4.33	4.35
40	4.37	4.40	4.42	4.45	4.48	4.50	4.53	4.55	4.57	4.59
50	4.61	4.63	4.65	4.67	4.69	4.71	4.72	4.74	4.75	4.77
60	4.78	4.80	4.81	4.83	4.84	4.86	4.87	4.89	4.91	4.93
70	4.94	4.95	4.97	4.98	4.99	5.01	5.02	5.04	5.05	5.07
80	5.08	5.09	5.10	5.12	5.13	5.14	5.15	5.16	5.17	5.18
90	5.19	5.20	5.21	5.22	5.23	5.24	5.25	5.26	5.27	5.28
100	5.29									
200	5.99									

Example two:

Condition II, where T_u is greater than h but less than 3h.

Given:

h = 5 feet

r = 0.125 feet

Q = 0.10 gallons per minute

T_u = 6 feet (greater than h but less than 3h; use equation 10)

T = 18° C.

$$k = \frac{192 \log_e \left(\frac{h}{r}\right) \frac{Q}{2\pi}}{h^2 \left(\frac{1}{6} + \frac{T_u}{3h}\right)} C_t \qquad (10)$$

$\log_e 40 = 3.69$

$$k = \frac{192 \text{ X } 3.69 \text{ X } \frac{0.10}{6.28}}{25 \quad (\frac{1}{6} + \frac{6}{15})} \text{ X } 1.05 = 0.77 \text{ ft}^3/\text{ft}^2/\text{day}$$

The natural logarithms of h/r are obtained from table 2-3.

Example three:

Condition III, where T_u is less than h:

Given:

h = 5 feet

r = 0.125 feet

Q = 0.10 gallons per minute

T_u = 3 feet (less than h; use equation 11)

T = 18° C

$$k = \frac{192 \log_e \left(\frac{h}{r}\right) \frac{Q}{2\pi}}{h^2 \left(\frac{T_u}{h} - \frac{T_u^2}{2h^2}\right)} C_t \qquad (11)$$

$$= \frac{192 \times 3.69 \times \frac{0.10}{6.28}}{25\left(\frac{3}{5} - \frac{9}{50}\right)} \times 1.05 = 1.13 \text{ ft}^3/\text{ft}^2/\text{day}$$

Soil-sampling Tools

General

A wide variety of cutting and sampling tools are available on the market. Tables 2-7 and 2-8 may be used as a guide in selecting tools for drilling, logging, and sampling purposes in various soils. Only those logging and sampling tools recommended for SCS use are discussed in this Handbook.

Open-drive and Piston Samplers

Open-drive samplers are cylindrical samplers which are pushed or driven into the materials to be sampled. A drive sampler equipped with a piston is known as a piston sampler. A large number of drive and piston samplers are available on the market. They are manufactured in a variety of diameters, tube thicknesses and tube lengths. They are generally known as thick wall, thin wall, and split barrel samplers.

Thin-wall Open-drive Samplers

Thin-wall open samplers consist of solid thin-wall barrels. These are manufactured in a variety of lengths, diameters, and wall thicknesses. They must be equipped with ball or other types of check valves for satisfactory performance.

The simplest type of open drive sampler is the so-called thin-wall "Shelby Tube," Figure 2-14. It should be obtained in steel tubing lengths of 24 inches and from 3½ to 5 inches in diameter. The tube is attached to a head assembly by means of set screws. This head assembly contains a ball check valve. After the sample is obtained, the tube is detached from the head, sealed, and shipped to the laboratory where the sample is removed for conducting tests.

Thin-wall samplers do not have cutting shoes but rather a sharpened cutting edge. To provide clearance in certain materials, the edge may be swaged to cut a sample smaller than the inside diameter. Table 2-9 lists recommended bit clearance for different types of material.

Thin-wall drive-samplers provide good undisturbed samples of certain soil materials if proper methods of operation are used. The sampler must be advanced by a uniform and uninterrupted push without rotation. No additional drive should be made after the sampler stops. This requires that the drill rig be provided with a hydraulic pressure device capable of exerting a driving force of at least 8000 pounds. Since the drill rig serves as a reaction for driving the sampling tube, it may be necessary to anchor the rig to hold it down.

Sampler	Minimum Diameter 1/ Required for:						Materials in Which Used
	Hole advancement	Logging	Consolidation Tests 2/	Direct Shear	Triaxial Shear	Horizontal Permeability	
Continuous Helical Augers	3 O.D.						Medium soft to stiff cohesive soils free of cobbles and boulders. Unsaturated but wet sand and silt.
Iwan Hand Augers	2 O.D.	2 3/					
Closed Bucket Augers	3 O.D.	3 O.D. 3/					All, including gravel. Free of cobbles and boulders.
Slat-type Bucket Augers	3 O.D.	3 O.D. 3/					Cohesive soils.
Split-barrel		1-3/8 4/					All but hard and brittle soils free of coarse gravel, cobbles and boulders.
Dry-barrel	3 O.D.	3 O.D.					Cohesive soils.
Thin Wall		3	3	3	3-5 5/	5	Soft to stiff and loose to medium.
Stationary Piston-Thin Wall		3	3	3	3-5 5/	5	Same as above but includes very soft and very loose soils
Chopping-Jetting	2 O.D.						Stiff to hard clays, brittle soils, dense sand, partially cemented soils. All but very soft soils.
Double Tube Soil Core Barrel (Denison)	4 O.D.	2-15/16	2-15/16	2-15/16	3-5 5/	5	
Roller Bit	3 O.D.						Hard to soft rock.
Double-Tube Rock Core Barrel	2-15/16 O.D.	2-1/8	2-1/8	2-1/8	2-1/8	2-1/8	

1/ Applies to inside diameter unless indicated otherwise.
2/ Includes vertical permeability tests.
3/ Recommended only for use in homogeneous materials.
4/ Also standard penetration test.
5/ Three-inch samples are suitable when foundation materials are relatively homogeneous. Five-inch samples are required when stratification of the profile is significant.

Table 2-7 Recommended Logging and Sampling Tools, with Minimum Diameter

Table 2-8 Soil Types and Sampling Tools

Type of Soil	Logging or Disturbed Samples	Undisturbed Samples
Common cohesive and plastic soils.	Bucket-type augers,[1/] all types of drive samplers, dry barrel.	Thin-wall open-drive sampler. Piston sampler. Double-tube core barrel
Slightly cohesive and brittle soils including silt, loose sand above the water table.	Same as above.	Thin-wall open-drive samplers. Piston samplers below water table. Double-tube soil core barrel (with liner).
Very soft and sticky soils.	Closed bucket auger,[1/] dry barrel, piston sampler or open drive with core retainers.	Thin-wall or piston samplers.
Saturated silt and loose sand.	As above. Overdrive push-tubes to retain sample.	Piston sampler with heavy mud.
Compact or stiff and brittle soils including dense sands, partially dried soils.	Bucket-type auger.[1/] Thick-wall drive sampler.	Double-tube soil core barrel.
Hard, highly compacted or partially cemented soils, no gravel or cobbles.	Bucket auger.[1/] Thick wall drive sampler and hammer. Double-tube core barrel	Double-tube soil core barrel.
Coarse, gravelly and stony soils including compact and coarse till.	Bucket auger[1/] [2/] Large diameter thick wall drive sampler.	Not practical. (Advance freezing and core.)
Organic clay, silt or sand.	As above according to basic soil type.	Thin-wall piston. Measure length of drive and original volume of sample carefully.

1/ Homogeneous soils only.

2/ Power equipment such as bulldozers and backhoes are more suitable in many cases.

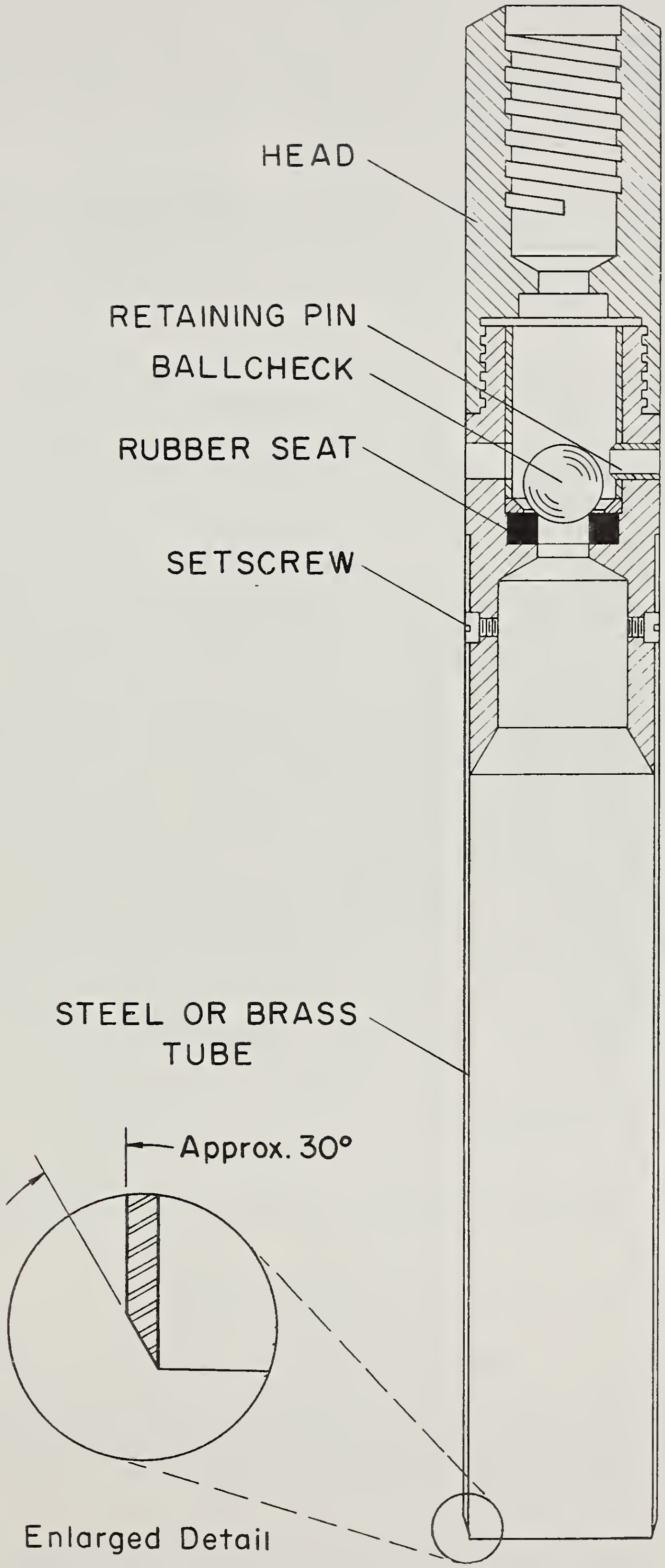


Figure 2-14 Thin-wall Open Drive Sampler

Thin-wall drive sampling methods are most practical in fine-grained, plastic, or peaty soils. The method is not suited for sampling brittle, cemented, or gravelly soils. The amount of disturbance in drive samples depends to a great extent on the dimensions of the sample tube. The thinner the wall and the larger the diameter, the less the disturbance will be.

Extruded samples are excellent for logging purposes. If desired for this purpose, the drill rig must be equipped with a sample ejector. The sample should be extruded through the top of the sampler.

Undisturbed samples for laboratory analysis should not be removed from the sampling tube in the field, but should be sealed in the tube and the tube sent to the laboratory. Either 3-inch or 5-inch diameter undisturbed samples should be taken depending upon the type of laboratory test desired.

Three-inch diameter tubes usually have a wall thickness of 1/16 inch and 5-inch diameter tubes about 1/8 inch. These tubes have no cutting shoe. Equipment is available to swage thin-wall tubes to provide desirable bit clearance. Bit clearance refers to the difference between the diameter of the cutting edge and the inside diameter of the sampling tube, in percent. For example, if the cutting edge is 2-15/16 inches and the I.D. of the sampling tube is three inches, then the bit clearance is 3 minus 2-15/16 divided by 3 (I.D.), or about 2 percent. Bit clearance may not be necessary if use of the sampler is limited to soft, loose, non-cohesive materials. Tubes with the proper bit clearance should be available at the site to obtain adequate undisturbed samples for each type of material encountered. Cohesive soils and soils which are slightly expansive require varying amounts of bit clearance. Saturated and soft, loose sands, silts and some clays may require little or no clearance. The clearance selected is that necessary to minimize drag or sidewall friction on the sample and to assist in retaining the sample in the tube.

Thin-wall sampling procedure.--The basic procedures for thin-wall tube sampling are as follows:

1. Advance hole to strata to be sampled.

2. Clean hole thoroughly to remove loose material from the bottom.

3. Be sure drilling rig is anchored or heavy enough to counteract the pressure it is capable of exerting. This should be at least 8000 pounds.

4. Sample tube must be smooth, thoroughly cleaned inside and out-side, be properly sharpened, and have correct bit clearance for the type of soil being sampled. (See Table 2-9). Tubes should be plastic coated or greased to prevent rusting.

Table 2-9 General recommendations for thin-wall drive sampling

Soil type	Moisture condition	Soil consistency	Length of drive, inches	Bit clearance, percent	Open-drive sampler recovery	Recommendations for better recovery
Gravel		(Thin-wall drive samplers not suitable)				
Sand	Moist	Medium	18	0 to $\frac{1}{2}$	Fair to poor	
Sand	Moist	Loose	12	$\frac{1}{2}$	Poor	Recommend piston sampler.
Sand	Saturated	Medium	18	0	Poor	Recommend piston sampler.
Sand	Saturated	Loose	12 to 18	0	Poor	Recommend piston sampler.
Silt	Moist	Firm	18	$\frac{1}{2}$	Fair to good	
Silt	Moist	Soft	12 to 18	$\frac{1}{2}$	Fair	
Silt	Saturated	Firm	18	0	Fair to poor	Recommend piston sampler.
Silt	Saturated	Soft	12 to 18	0 to $\frac{1}{2}$	Poor	Recommend piston sampler.
Clay and shale	Dry to saturated	Hard	(Thin-wall drive sampler not suitable)			Recommend double-tube sampler.
Clay	Moist	Firm	18	$\frac{1}{2}$ to 1	Good	
Clay	Moist	Soft	12 to 18	1	Fair to good	
Clay	Saturated	Firm	18	0 to 1	Good	
Clay	Saturated	Soft	18	$\frac{1}{2}$ to 1	Fair to poor	Recommend piston sampler.
Clay	Wet to saturated	Expansive	18	$\frac{1}{2}$ to $1\frac{1}{2}$	Good	

Adapted from U.S.B.R. Earth Manual, 1960.

5. Attach tube to drill rod and lower into hole until it rests on the bottom.

6. Drive must be made without rotation and with one continuous stroke.

7. Length of drive must be carefully measured and should be a few inches short of the sampler length to prevent compaction of the sample in the tube by over driving. See Table 2-9.

8. Rotate sampler slightly to break off sample before pull is started.

9. Retrieve sampler from hole carefully and with a steady pull to avoid sample loss.

10. Length of sample recovered must be accurately measured and recorded.

11. Samples are sealed in tube for handling and shipping.

<u>Piston-drive Samplers</u>
Piston samplers are thin-wall samplers similar to "Shelby" thin-wall samplers but containing a piston to facilitate sampling. It is designed to obtain samples of soft or medium soils and for obtaining samples of sands, silts, and cohesive soils below the water table. See Tables 2-7 and 2-8. The stationary-piston sampler (Figure 2-15) is lowered to the bottom of the bore hole with the piston held in the lower end of the sampler. The piston is then locked into position by means of actuating rods which extend to the surface within the drill rods. The tube is then forced into the materials by steady pressure, while the piston remains stationary at constant elevation, to obtain the sample. The sampler is equipped with a vented head to permit escape of air above the piston. The piston creates a vacuum which holds the sample in the tube while it is being brought to the surface.

Stationary-piston samplers are available in sizes up to 30 inches in length with I.D. up to 4-3/8 inches. A modification of the above sampler (Osterberg type) requires lowering of the sampler in the bore hole and forcing the sampling tube into materials by means of hydraulic pressure applied through the drill rods. This type of sampler is available in 3-inch and 5-inch diameters. This type of sampler is recommended for those soils requiring a piston sampler.

Piston sampling procedure--The basic principles of operation for stationary-piston sampling are the same as for thin-wall sampling with the exception of techniques for locking the piston which vary with the type of sampler. Additional considerations are as follows: (1) The hydrostatic pressure of drilling fluids aids the suction effect of the piston. The consistency of the mud should be such

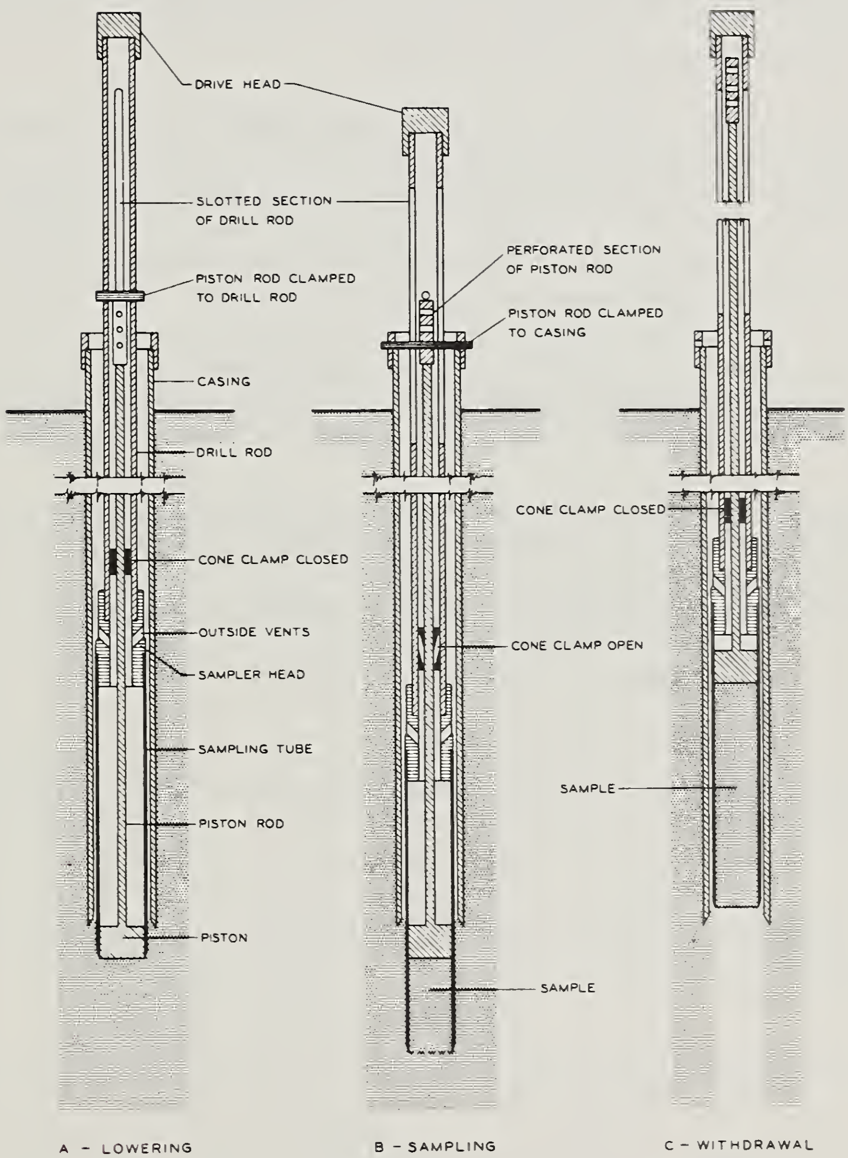


Figure 2-15 Stationary-Piston Sampler

that about ½ inch of the sample is saturated with mud; and, (2) tubes have little or no bit clearance and no sample retainers.

The basic procedures for sampling with the stationary piston sampler are as follows:

1. Advance and clean hole to sampling horizon as in open thin-wall drive sampling.

2. Attach sampler to drill rod.

3. Insert the piston and lock flush with the bottom of the tube.

4. Where a piston rod is used, it is connected to the piston either after the sampler has been lowered to the bottom of the hole or it may be connected piece by piece inside the drill rod while the sampler is being lowered.

5. The sampler is placed at the bottom of the bore hole and the piston unlocked by turning the piston rod clockwise five revolutions (this may vary with different samplers.)

6. The piston rod is then secured to the drill rig, casing, or preferably to an independent frame and the sampler injected under steady pressure into the material to be sampled. Weight of equipment is normally used as the reacting force.

7. Carefully measure length of drive.

8. At the end of the drive the piston rod is disconnected by continuing to turn it clockwise until the rod is fully released, then the sampler is raised. The piston is held in place at the top of its stroke by a split-cone clamp.

9. Before removing the sampler from the drilling fluid at the ground surface, the hand or a block should be placed over the end to prevent the sample from dropping out.

10. Remove sampling tube from drill rod and measure and record length of sample.

11. Seal sample in tube for shipment to the laboratory.

<u>Split-barrel Sampler</u>
The split-barrel sampler (Figure 2-7) consists of a head, barrel and cutting shoe. The barrel is split longitudinally so that it can be taken apart, after removal of the head and the shoe, and the sample removed for visual inspection or packing in jars or other containers for shipment to the laboratory. The split-tube sampler can withstand hard driving into soil materials. Since

cutting shoes often become damaged by driving, a supply of additional cutting shoes should always be available in the field.

Split-tube samplers may be obtained in lengths up to 24 inches. The 2-inch O.D. sampler is recommended for logging purposes and is required for the standard penetration test. It is not suitable for taking an undisturbed sample because of sample disturbance due to the thick cutting shoe and driving action of the hammer. Split-barrel samplers are adapted for accurate logging of thin-bedded materials.

Split-barrel sampling procedures--Sampling and standard penetration tests are normally run in conjunction with each other. Procedures for sampling and running standard penetration tests are as follows:

1. Advance and clean out hole to horizon to be logged.

2. Attach split-barrel sampler to drill rod and lower to bottom of hole. Sampler must be 2 inch O.D. and 1 3/8 inches I.D. of the cutting edge for standard penetration test.

3. Drive sampler 6 inches into strata and mark drill stem with chalk to provide for 1-foot drive.

4. With 140-pound drive hammer, falling 30 inches, drive sampler for distance of 12 inches. Count and record number of blows required for each 6 inches of this 12-inch test.

5. Rotate sampler slightly to break off before pull is started and retrieve sampler.

6. Remove sampler from rod and disassemble by removal of cutting shoe, head, and separating the two halves of the barrel.

7. Carefully log sample and record length.

8. Place samples needed for laboratory analyses in sealed jars to retain natural moisture.

9. Clean, inspect, and reassemble sampler for next drive. If cutting shoe has been damaged by the previous drive to the extent that the inside diameter is appreciably altered, it must be replaced with a proper cutting shoe.

<u>Double-Tube Soil Core Barrel Sampler (Denison Type)</u>
The most satisfactory sampler for obtaining nearly undisturbed soil samples of highly compacted, hard, stiff, uncemented or slightly cemented materials is the double-tube soil core barrel

with liner (Figure 2-16). Samples of cohesive soils are obtained with a double-tube soil core barrel with the least amount of disturbance. Double-tube samplers can be used to sample a wide variety of materials including some rock such as soft shales and soft and friable sandstones. The method is not satisfactory for obtaining undisturbed samples of soft, loose, cohesionless silts and sands below the water table, or very soft and plastic cohesive materials where the structure is destroyed by barrel whip. It is not suitable for obtaining undisturbed samples of gravels and cobbles.

The double-tube core barrel is advanced by rotating the outer barrel which cuts a circular groove and loosens the soil material to be displaced by the two barrels. Drilling fluid is forced downward through the drill stem between the barrels and carries the cuttings to the surface outside the tubes and drill stem. The inner barrel which does not rotate, moves downward over the undisturbed sample being cut by the rotating outer barrel. A liner is inserted in the inner barrel before the barrel is assembled. After drilling the required length, the sampler is withdrawn and the liner removed and prepared for shipping.

The outer cutting bit or the inner cutting shoe of double-tube soil core barrels are made in several lengths so that the relation of the cutting edge of the inner barrel to the bit can be varied. A retracted inner shoe or long outer bit is used for very hard soils which are not subject to erosion. In dense or brittle soils, a short bit is used so that the inner barrel is nearly flush with the cutting teeth. Soft, loose, or slightly cohesive soils require the shortest bit and the maximum protrusion of the inner barrel so that the drilling mud does not wash out, penetrate, or undercut the sample. The sample should enter the barrel so that it fills the liner but the outer barrel should cut the core so that a minimum of downward pressure is required. The number of teeth of cutting bits varies from 6 to 24, their height from 1/8 to two inches, and the outward projection from 1/16 to 1/2 inch. Systematic experimentation to determine the optimum number, shape, and dimension of the teeth for various types of soils is needed, and consideration may be given to teeth with the cutting edge or face at an angle with the radius, so that the teeth will tend to carry the cuttings toward the outer rim of the bit. Often blank bits are furnished by the manufacturer. These are cut, shaped, and built up with a hard alloy as desired.

Basket or spring-type core retainers may be used. Several types, using a different number and flexibility of springs, are available for use in different materials. The tapered, split-ring core retainer used rock core barrels is not satisfactory for use in soil. A check valve is provided to relieve pressure over the core. The coring bits used usually have hard surfaced steel teeth.

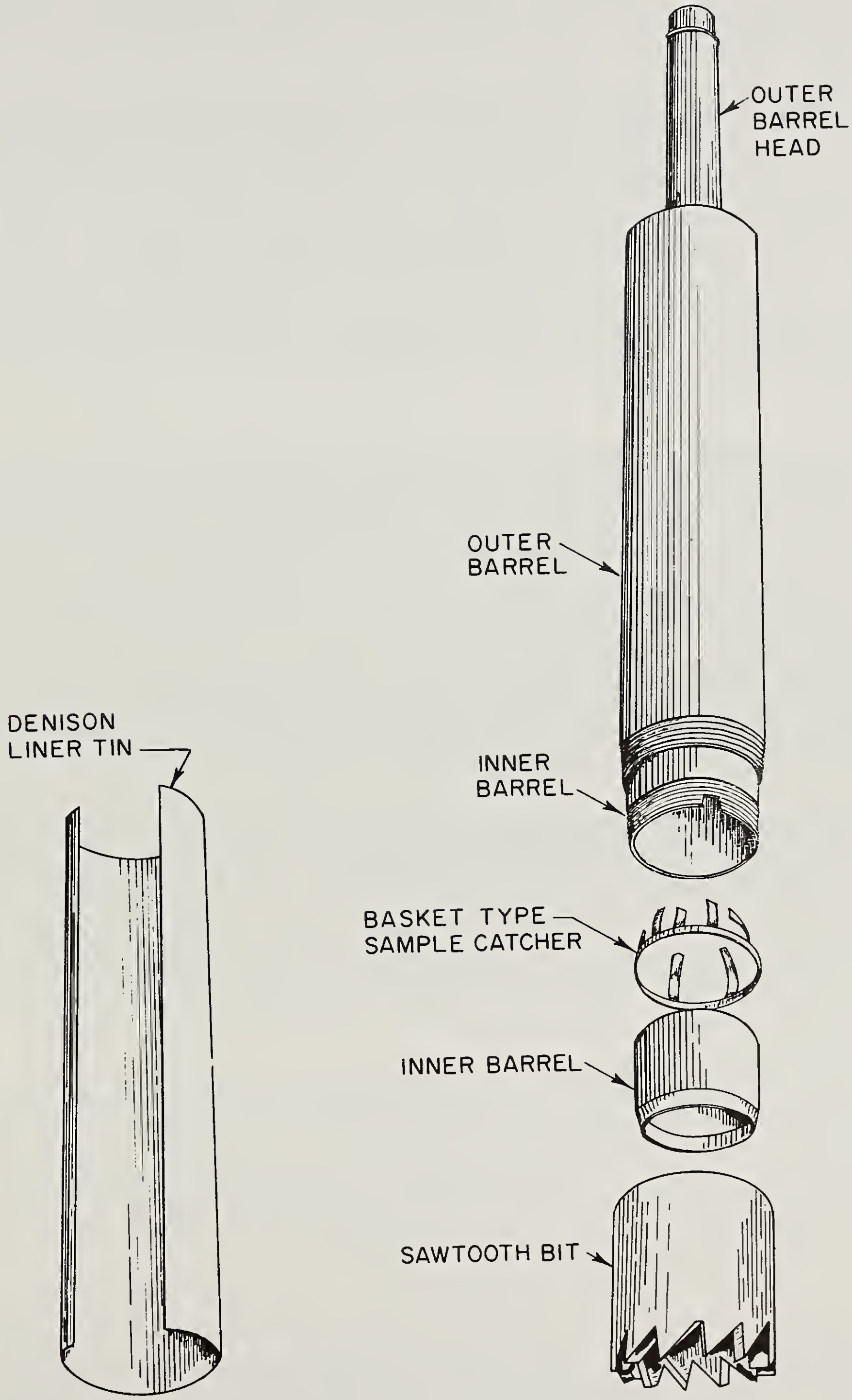


Figure 2-16 Double-tube Soil Core Barrel (Denison)

Double-tube soil core barrels with liners come in various sizes which obtain untrimmed soil samples ranging from 2-3/4 inches to 6 inches in diameter. The diameter of undisturbed core needed depends upon the kind of laboratory test required. Core barrels which obtain undisturbed samples of about two feet in length are recommended.

Sectional liners are recommended for use in denison-type core barrels when taking undisturbed samples for laboratory analysis. They should be made of stainless steel, preferably seamless, of cylindrical shape. Sectional liners, 8 inches long and 1/32 inch thick, are available from equipment manufacturers. A welded side joint is satisfactory if seamless construction is not available. Samples may be extruded at the laboratory from such liners with a minimum degree of disturbance and the liners returned to the field for reuse.

When sectional liners are used, three sections are taped together and placed in the barrel. The tape facilitates removing the sections from the barrel after sampling and also prevents water from entering between the sections and washing away the sample. The I.D. of the liners must be 1/8 inch smaller than the I.D. of the inner barrel on the sampler to be used, to allow clearance for the tape. A strong tape is needed to tape the sections together. GSA No. 8135-582-4772 is recommended. This type has longitudinally aligned filaments in the adhesive and has a tensile strength of 300 pounds per inch of width.

Two-foot, 20-gage galvanized metal liners may also be used to take samples for laboratory analysis. They should have welded and soldered seams. Split-tin liners or liners with loosely crimped seams are satisfactory for field logging purposes. The I.D. of the shoe on the inner should be machined or swaged down to 1/16 inch less than the I.D. of the liner, so that the sample will easily enter the liner.

Bit speed--Operational procedure in soil coring must be determined by trial for each soil condition. The rate of penetration, the speed of rotation, the length of cutting bit, the consistency of the drilling mud, and the pump pressure are all dependent upon soil conditions. The speed of rotation for soils and soft rock may vary from 40 to 125 rpm.

Bit pressure--The pressure on the coring bit and its rate of advance or feed must be carefully adjusted in accordance with the character of the material encountered, type of bit, and bit speed. Too high a bit pressure and rate of feed may damage the bit and cause plugging of the bit and fluid passages and failure of the sample before it enters the barrel. Too low a bit pressure and slow or intermittent feed may expose the core to excessive erosion and torsional stresses. As the sample enters the inner barrel, wall friction increases, and bit pressure must be increased

to maintain a constant rate of advance. Generally, the rate of penetration should be no greater than the speed at which the outer barrel is able to cut.

A technique which assists in recovering a double-tube sample in materials of low cohesion is to shut off the circulating pump about two inches before the end of the drive. With the circulation shut off, the coring operation is completed. The soil will be forced up between the inner and outer barrels. This causes the inner barrel to turn with the outer barrel and shears off the core. Consequently, there is no tendency for the sample to be pulled out of the barrel by material to which it is attached at the base when the sampler is removed from the hole. This procedure will also result in compaction in the inner barrel shoes and form a plug which will assist in sample recovery. The core catcher normally is not required when this technique is used.

Double-tube soil core barrel sampling procedures-- The basic rules of operation for double-tube soil core barrels are as follows:

1. Assemble barrel, attach to drill rod, and lower to bottom of hole.

2. Initiate drilling mud circulation and rotation of barrel. Both bit speed and rate of drilling mud circulation should be increased slowly to their optimum rates.

3. Length of drive must be carefully measured. Total drive length should be a few inches shorter than the sample container length to prevent overdrive.

4. Downward force should be a minimum and regulated to that speed at which the outer barrel is able to cut.

5. The rotation of the bit should be limited to a speed which will not tear or break the sample.

6. The consistency of the drilling mud should be thick enough to prevent caving of the hole.

7. Pump pressures should be the minimum required to carry the cuttings from the hole.

8. The sampler should be withdrawn from the hole carefully so as not to disturb the core during withdrawal.

9. Disassemble barrel and remove sample liner with sample.

10. Carefully remove all disturbed materials from both ends of the liner.

11. Seal ends for shipment to the laboratory.

Rock-sampling Tools

Double-Tube Rock Core Barrel Sampler

General--Rock core barrels are of two types, single tube and double tube. The single tube is designed primarily for boring in sound rock or for taking large cores in all types of rock. Double-tube rock core barrels are particularly useful for drilling small holes in sound rock, for drilling fissured rock, and for drilling soft rock where the core needs to be protected from the erosive action of drilling water. The fluid passes between the inner and outer barrels eliminating its erosive action. There are two types of double-tube core barrels, "rigid" and "swivel." In the rigid type the inner tube and outer tube rotate together while in the swivel-type double-tube core barrel, the inner tube does not rotate. The double-tube rock barrel (Figures 2-17, 2-18) differs from the soil-coring barrel in that it does not have a removable liner to hold the sample, and in the relationship of the cutting shoe to the inner shoe. The cutting shoe trims the core at slightly less than that of the inner barrel and the sample is retained in the inner barrel by means of a core catcher. The rock core barrel obtains a sample of rock in the shape of a cylindrical core. The circular bit cuts the core and the barrel slides down over it. A ball-check valve to relieve water pressure, and a core catcher assist in retaining the core in the barrel. Table 2-10 lists the various sizes of coring bits and barrels used in SCS work. A double-tube swivel-type core barrel in NX size is recommended for the types of rock boring generally required in SCS work. With some types of drilling rigs a short barrel, taking a one-foot core, is useful for starting a bore hole where rock outcrops at the surface and cores are desired from the surface down. (Figure 2-18)

Rock core barrels may have either the "retracted inner barrel" or the "bottom discharge bit." In the retracted inner barrel the drilling water passes down between the inner and outer barrels, across the cutting teeth of the metal bit or the waterways of the diamond bit and up outside the outer barrel. It is used in non-erodible rock. In the bottom discharge bit the drilling fluid passes out through holes in the bottom of the bit proper, and thence up outside the outer barrel. It is used mainly in soft or broken rock.

Rock core bits--In general, a small number of relatively long teeth are preferable for coring soft rock formations such as those containing clay streaks or shale layers. On the other hand, a large number of small teeth provides a greater rate of progress and causes less disturbance of the material when coring in medium-hard formations.

Steel saw-tooth coring bits are usually provided with teeth or inserts made of very hard and abrasion-resistant tungsten carbide alloys which are sold under various trade names. Blank bits are also available for customers who desire to set their

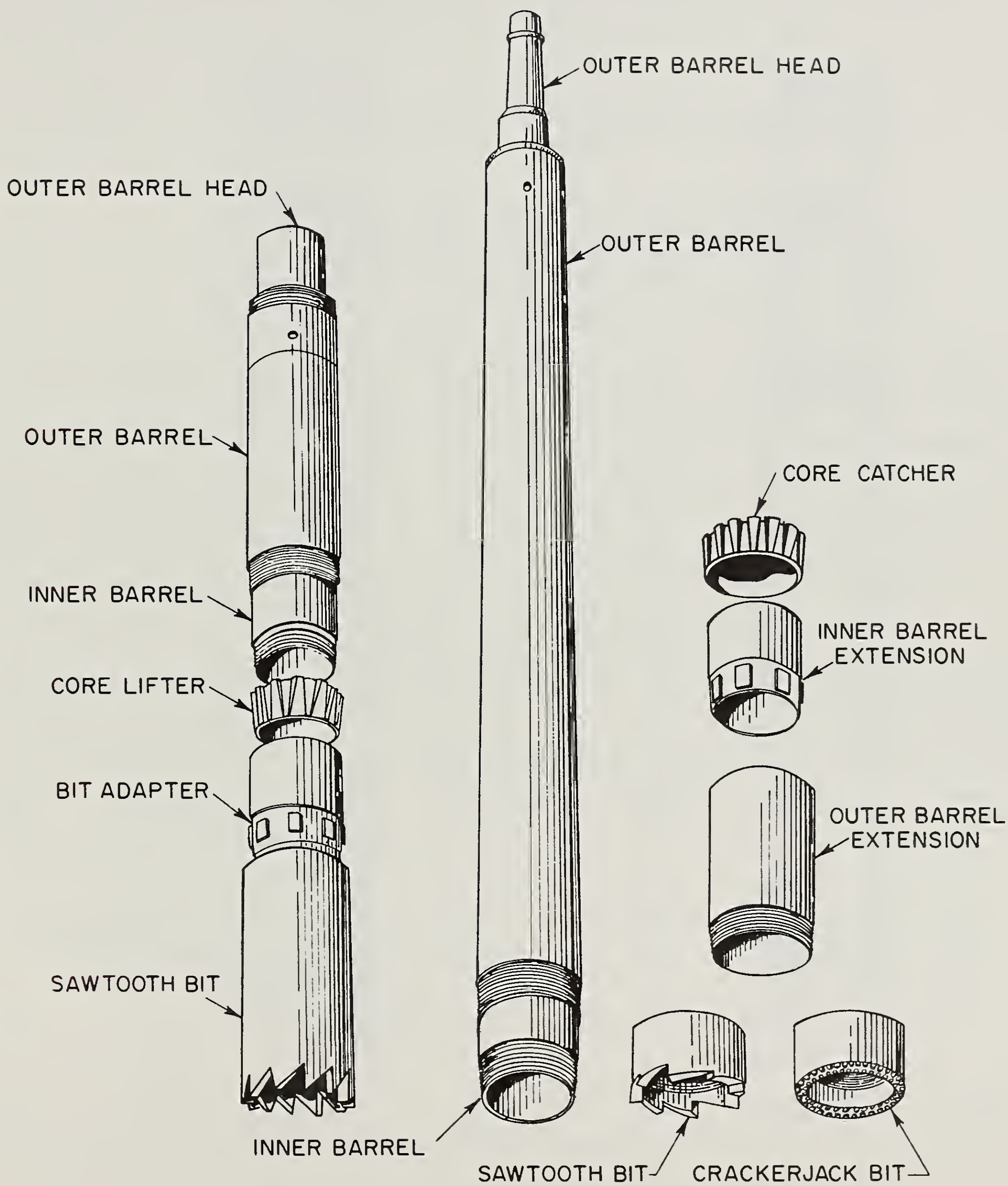


Figure 2-17 Long and Short Rock Core Barrel

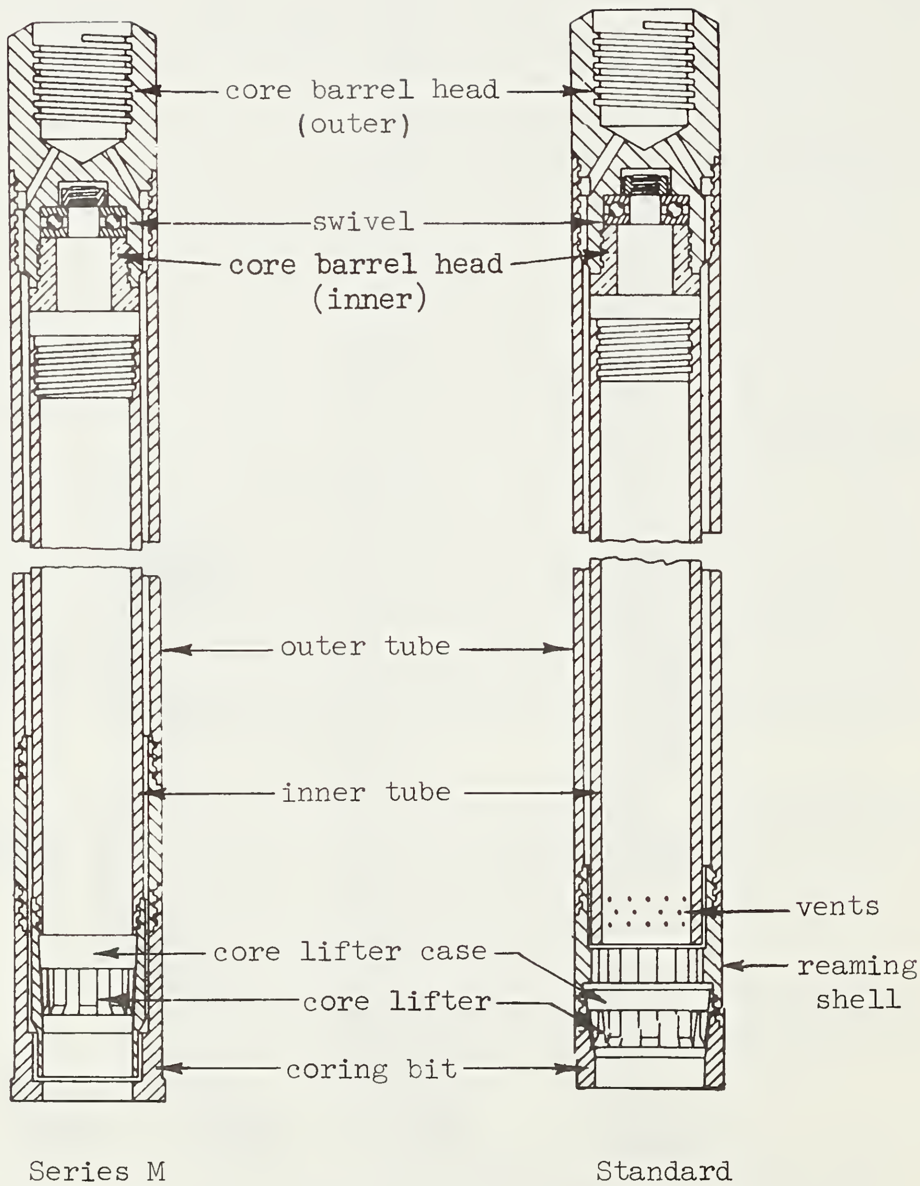


Figure 2-18 Swivel-type Double-tube Rock Core Barrels.

Table 2-10 Standard Sizes of Coring Bits and Barrels, Casing and Drill Rod.

ROCK AND SOIL CORING BITS AND BARRELS

Size Inches	Hole Inches	Core Inches
AX or AWX	1-7/8	1-1/8
BX or BWX	2-3/8	1-5/8
NX or NWX	2-15/16	2-1/8

FLUSH COUPLED CASING AND COUPLING

CASING				COUPLING			
Size	O.D. Inches	I.D. Inches	Wgt. lbs. per ft.	Size	O.D. Inches	I.D. Inches	Wgt.
AX	2-1/4	2	2.9	AX	2-1/4	1-29/32	1.8
BX	2-7/8	2-15/32	5.7	BX	2-7/8	2-3/8	3.2
NX	3-1/2	3-1/16	7.8	NX	3-1/2	3	4.0

DRILL ROD				DRILL ROD COUPLING			
Size	O.D. Inches	I.D. Inches	Wgt. lbs. per ft.	Size	O.D. Inches	I.D. Inches	Wgt.
A	1-5/8	1-1/8	3.7	A	1-5/8	9/16	1.7
AW	1-3/4	1-7/16	4.2	AW	1-3/4	5/8	2.2
B	1-7/8	1-1/4	5.0	B	1-7/8	5/8	2.4
BW	2-1/8	1-13/16	5.3	BW	2-1/8	3/4	3.1
N	2-3/8	1-5/8	5.2	N	2-3/8	1	4.0
NW	2-5/8	2-5/16	5.5	NW	2-5/8	1-3/8	5.5

(From The Diamond Core Drill Manufacturers Association)

own bits. Steel bits are much less expensive than diamond bits, but are recommended for use only in soft or moderately soft rock. The rate of progress in hard rock is slow with steel bits and the bits wear rapidly.

Diamond bits are used for coring all but the soft and moderately soft rock. They are of two kinds - set and impregnated. In set bits, industrial diamonds, usually West African Bortz, are set by hand or machine in a tungsten carbide matrix metal. Reaming shells serve to stabilize the bit and prevent it from whipping from side to side. Diamond bits and reaming shells may be used interchangeably with steel bits on the double-tube rock core barrels. When set diamond bits and reaming shells become worn, they are returned to the manufacturer for resetting. The remaining stones are salvaged in this process.

Normally it is advantageous to have at least three types of diamond (set) bits available: One for very hard formations (chert or quartzide), one for hard formations (sandstone or dolomite), and one for medium hard formations (cemented shale). Bits for harder formations use smaller stones, greater total weight of stones, and less water courses. Using each bit only in the formations for which it is designed results in much longer life for the bits. Typical diamond bits and reaming shells are shown in Figure 2-19.

In impregnated bits the entire cutting edge of the bit is impregnated with small industrial diamond fragments. They are especially recommended for very hard, broken formations, which might result in "shelling" of diamonds from a set bit. Sand-blasting is sometimes required to expose new cutting points on these bits. They cannot be reset and have no salvage value. They are used until completely worn out and then discarded.

Boring in loose, unconsolidated materials will seriously damage diamond bits. Where unconsolidated materials must be penetrated to reach rock, casing should be used and the hole thoroughly cleaned before boring with a diamond bit. Flush-coupled casing is usually used for this purpose.

A split-ring core catcher is usually employed to seize the core when the barrel is lifted; after the core is broken off the catcher holds it in the barrel.

The inner barrel may or may not have vents to permit escape of the water displaced by the core. Omission of the vents theoretically reduces friction because water in the inner barrel above the core must be forced out between the core and the barrel, lubricating it. On the other hand, the escaping fluid may cause erosion of soft rock cores if the vents are omitted.

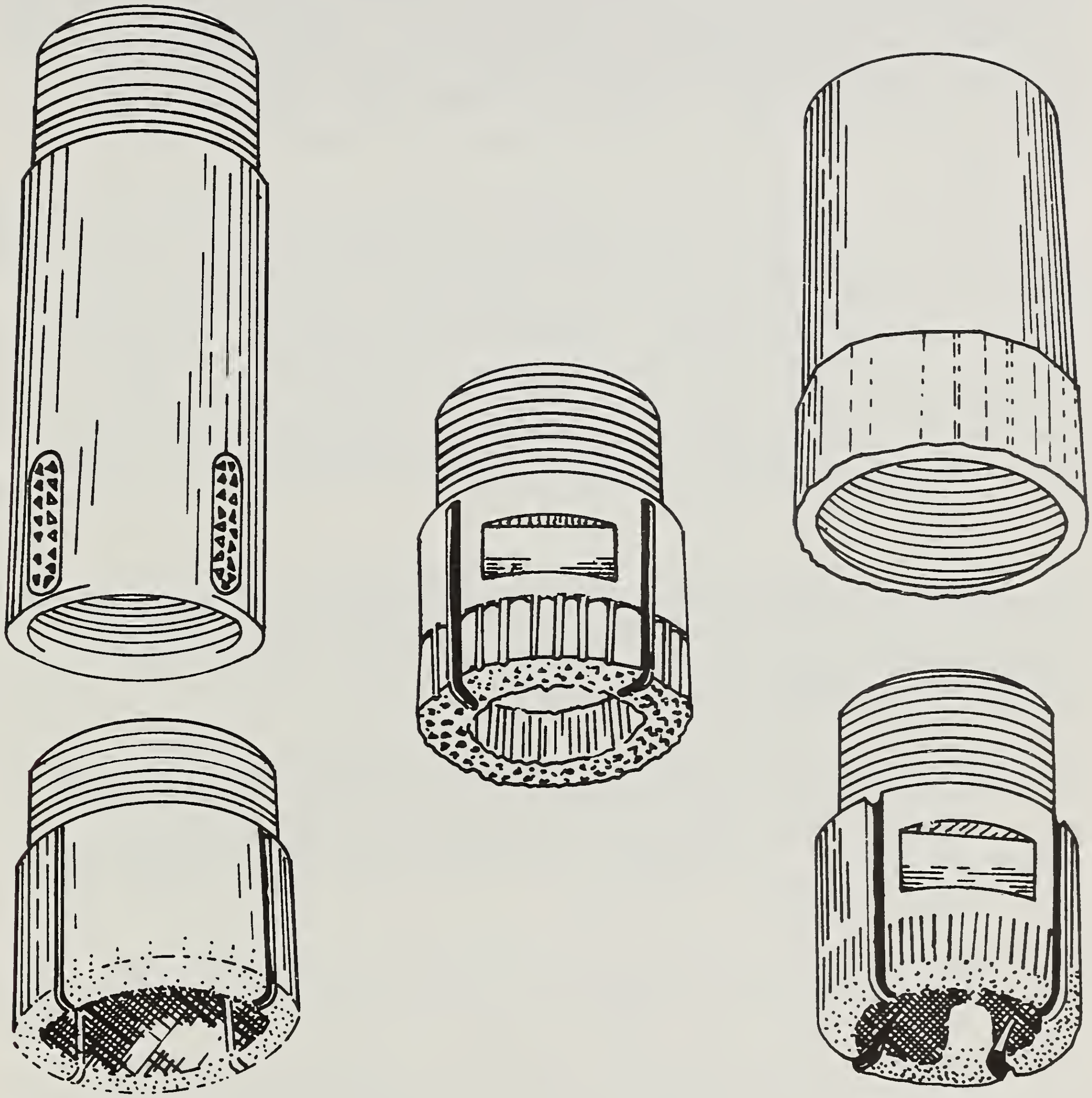


Figure 2-19 Diamond Bits and Reaming Shells

Rock core bit speed--Excessive bit speed will result in chattering and rapid wear of the bit and will break the core. A low bit speed results in a slow rate of progress and higher wear on diamonds. As the equipment becomes worn and the drill rods poorly aligned, it may be necessary to decrease the bit speed in order to avoid excessive vibration, whip, and chattering of the bit with consequent danger of breaking the core and damagine the bit. Higher speeds should be used in hard rock and lower in soft or broken rock. Diamond bit drilling speeds vary from 300 to 1500 rpm, while those for metal bits vary from 100 to 500 rpm.

Rock core bit pressure--The rate at which the coring bit is advanced depends upon the amount of bit pressure used. This pressure must be carefully adjusted to the character of the rock, the type of bit, and the bit speed. Excessive bit pressures, expecially in soft rock, will cause the bit to plug and may possibly shear the core from its base. The bit pressure is controlled by a hydraulic feed on the drilling machine. For shallow drilling, the weight of the drilling column will never exceed the optimum bit pressure except in very soft rock.

In hard rock a high feed pressure on diamond bits not only increases the rate of advance but also keeps the bit sharp and free cutting. A low feed pressure in the same rock tends to polish the diamonds. In softer rock the best results may be obtained with relatively low feed pressures.

Drilling fluids--Clear water is generally preferred to drilling mud in rock coring. Water requires smaller fluid passages and pump pressures, and the rock cuttings are generally fine and easily carries to the surface. In some cases air circulation is used with coring.

Where water is used, the flow of the return water should be regulated so that the cuttings are just carried out of the hole. If too much is used, the velocity across the bit face is increased which results in bit erosion. The metal holding the diamonds in a bit may be scoured away, leaving the diamonds exposed and easily pulled out. The flow should be increased when softer rock is being cored and larger sized cuttings are formed. However, it must not be great enough to wash away the core.

If water pressure begins to build up, there is generally some restriction at the bit face or in the core barrel. "Plugging" may occur where the rock is fractured or contains many seams. In this event a piece of rock wedges in the bottom of the core barrel, but also restricts water flow. The barrel may sometimes be cleared by lifting off the bottom a few inches and letting the hole wash. Sometimes it is best, if plugging is a problem, to drill through seams of broken rock with a roller bit. This method has the disadvantage, however, that no core is obtained.

If a diamond bit is run without sufficient cooling water (or air circulation), it will rapidly burn and stick fast and may be ruined.

A serious condition frequently encountered is the loss or "running off" of return water into cavities in the rock, causing what is known as a "blind hole." This condition can often be corrected by adding leaves, grass, or cotton hulls to the water. Such materials will often seal seams and cavities in the sides of the hole.

Hole cleaning--The hole should always be washed before starting to drill, as well as before pulling out the rods and tools. It should be kept in mind that at the end of a run there is a column of water extending the full length of the hole carrying the cuttings in suspension. If the pump is stopped immediately, these cuttings will settly to the bottom and when the bit is lowered it will rest upon a thick layer of mud instead of clean rock. The pump should be kept running until the return water is clear. When drilling through soft rock where there is a chance of washing the core away, the core barrel should be raised at least six inches before washing.

Core recovery--Percentage core recovery is an important factor in rock coring operations. The rock below the core barrel and in the lower part of the core is subject to torsion and vertical forces. The core is cut to a slightly smaller diameter than the inside diameter of the core barrel inner tube and, excepting cores of certain soft and swelling rocks, there is very little inside wall friction as long as the core is unbroken. However, inside friction develops when the core is broken and rock fragments become wedged between the core and inner barrel. A greater part of the feed pressure may then be transmitted to the core and to the rock directly below the core. The result is that weak sections of the rock, and possible all of it, is broken up and removed by the circulating water instead of entering the core barrel.

Since the torsional moment of resistance of the core increases with the cube of its diameter, an increase in diameter is very effective in reducing breakage and increasing the recovery ratio and the length of core obtainable. Normally the "NX" barrel (2-1/8-inch diameter core) is the smallest core barrel which should be used, mainly because of core recovery problems.

Other factors may also influence core recovery. Faulty core catchers may cause the loss of much core. Excessive water use may wash soft materials away. Warped drill rods or worn guides may result in undersize or broken and ground up core. The proper selection of bits, although primarily influencing the rate of drilling, has some application to core recovery, particularly in soft formations. For example, a steel bit is usually more effective in shale than a diamond bit, both in rate of

drilling and amount of recovery. Diamond bits generally have better recovery ratios than steel bits in harder rock.

Obviously, no core can be recovered when openings such as caverns, solution channels, or large open joints are encountered. The presence of such cavities can often be recognized by the drilling tools dropping several inches, suddenly, accompanied by a loss of water circulation.

In removing the core from the barrel it may be necessary to tap the barrel lightly with a hammer. The core should be laid out exactly as it comes out of the barrel, and allowances made for seams or fissures run through. The core barrel should be thoroughly washed and the joints lubricated before reassembling.

General considerations for rock coring--The following general considerations should be observed in rock coring work:

1. Always lower rods and tools carefully. Dropping an expensive diamond bit on the bottom of the hole can cause serious damage.

2. Don't let the drill bit bounce or vibrate on the formation.

3. Don't start the bit while it is resting on rock. Spin it into the formation slowly and cautiously at first. Increase the feed after penetrating four to six feet. The top rock is usually weathered, fractured, and seamy. Remember the old axiom among drillers that "more damage is done to the bit in the first five feet than in the next hundred." Also, the "solid rock" being drilled could be a boulder or projecting ledge, after which the bit could be ruined running through gravel and cobbles.

4. Don't slide a diamond bit over old core - spin over it. This will result in less diamond loss. A roller bit should be used if much core or caved material is left in the drill hole.

5. Don't throw a diamond bit into the tool box. Wash it off at the end of a run, disconnect it from the barrel, and put it away carefully. Never use a wrench on the diamond area. It is a diamond tool - treat it as such.

6. Don't force a diamond bit. The diamonds are set for maximum performance. If penetration rate in a uniform formation decreases, with bit speed and feed rate remaining the same, take off the bit. More damage is done to diamond bits through pressing to obtain another barrel of core than in any other way. If the bit is continued in service too long, the exposed diamonds are pulled out, and the loose stones riding around the bottom of the hole can quickly destroy the entire bit.

7. Check the rods and joints for leaks. Split or improperly connected rods can seriously reduce the circulation and cause overheating of bits.

8. Check the core barrels to make certain they are straight. Pay particular attention to the core barrel head.

9. Feed the drill with a steady pressure. Increasing or decreasing pressure and bit speed in a given formation normally will not increase rate of penetration but it will increase operating costs.

10. Keep an accurate record of the lengths of drill rods and tools in use. Trusting to memory can be expensive. In trying to remember odd lengths of drill rod, many a driller has become confused and dropped a string of tools 10 to 15 feet because he thought they were already on the bottom.

Tools for Advancing Bore Holes

Auger Bits

Many types of auger bits are available on the market. Helical or worm-type bits, sometimes called flight augers, are the most common. These are usually made in sections which may be added just as drill pipe is added. The augered material is brought to the surface by the helical action of the auger. A flight auger is useful for rapid advancement of holes. Since continuous flight augers mix materials throughout the hole, it is impossible to obtain a sample representative of one horizon. Where it is necessary to obtain a sample for logging or other purposes, augering should be stopped and a split-tube or other sampler substituted.

Other types of augers may have open or semi-closed sides. These have the advantage of less mixing of material than helical augers. The SCS has developed a set of auger bits, termed bucket auger bits, which are especially adapted for use with power augers for logging homogeneous materials and obtaining representative disturbed samples. These bits are shown in Figure 2-20. The open slat-type is adapted for use in cohesive materials, the closed cylindrical type in non-cohesive materials, and the semi-closed cylindrical type in intermediate materials.

The diameter of auger bit to use depends upon the purpose of the bore hole. The minimum diameter should not be less than three inches. Larger bits are required for relief wells. Auger bits may be used with either a rotary drilling rig or power auger. A different tool joint at the top of the auger bit, or a sub, may be required for use on different rigs.

Barrel Auger

The so-called "dry barrel sampler" (Figure 2-21) may be used with a core drill as a substitute for the auger bit. It is a completely closed cylindrical auger with no cutouts on the sides of the barrel. The cutting bit should have a slightly smaller inside diameter than the inside diameter of the barrel, to assist in removing the sample.

The sampler can be used as a substitute for an auger bit in cohesive soils. Water circulation is required for its use. The barrel is pushed into the soil with a rotary motion as with an auger bit. After the barrel has been removed from the hole, the sample is forced out and onto the sample catch pan by water pressure. This method of boring is much faster than dry augering with bucket augers in most soils, since it eliminates the time-consuming removal of materials from the auger bit by hand. As may be seen in Figure 2-21, the barrel is of very simple construction and can be made in any machine shop from casing, a modified casing shoe, and and adapter.

Chopping, Fishtail, and Jetting Bits

Chopping, fishtail, and jetting bits are used to detach and clean out material accumulated inside the drive pipe or casing. Water is pumped through the string of drill rods and discharged through ports at the face of the bit. Bits have different types of chopping edges designed for cutting into different types of materials. Hard steel alloys are commonly used to prevent excessive wear. Chopping, fishtail, and jetting bits come in a variety of shapes and sizes, the shape depending upon the type of material and size on the diameter of casing used. Normally a chisel or wedge-shaped bit is satisfactory for cohesive materials. Cross-chopping bits are designed for use in coarse gravel and boulders. Fishtail bits (Figure 2-22) are used for soft rock. A 2-5/8 bit is generally used in 3-inch casing. Large fishtail bits, 20 inches in diameter, often prove useful for installing relief wells.

Roller Bits

Roller bits usually have three sets of rollers with meshing, self-cleaning, hard-surfaced teeth. They are used for advancing holes in overburden and rock when no cores are required. This type of rock bit is commonly used in conjunction with augering to cut through rock after which augering is continued with a slightly smaller bit. The teeth are flushed by drilling fluid flowing out of vents in the base of the bit, and the cuttings are carried up through the hole.

Roller bits are manufactured in various sizes ranging from 2-3/8 inches to more than a foot. A tri-cone roller bit is shown in Figure 2-22

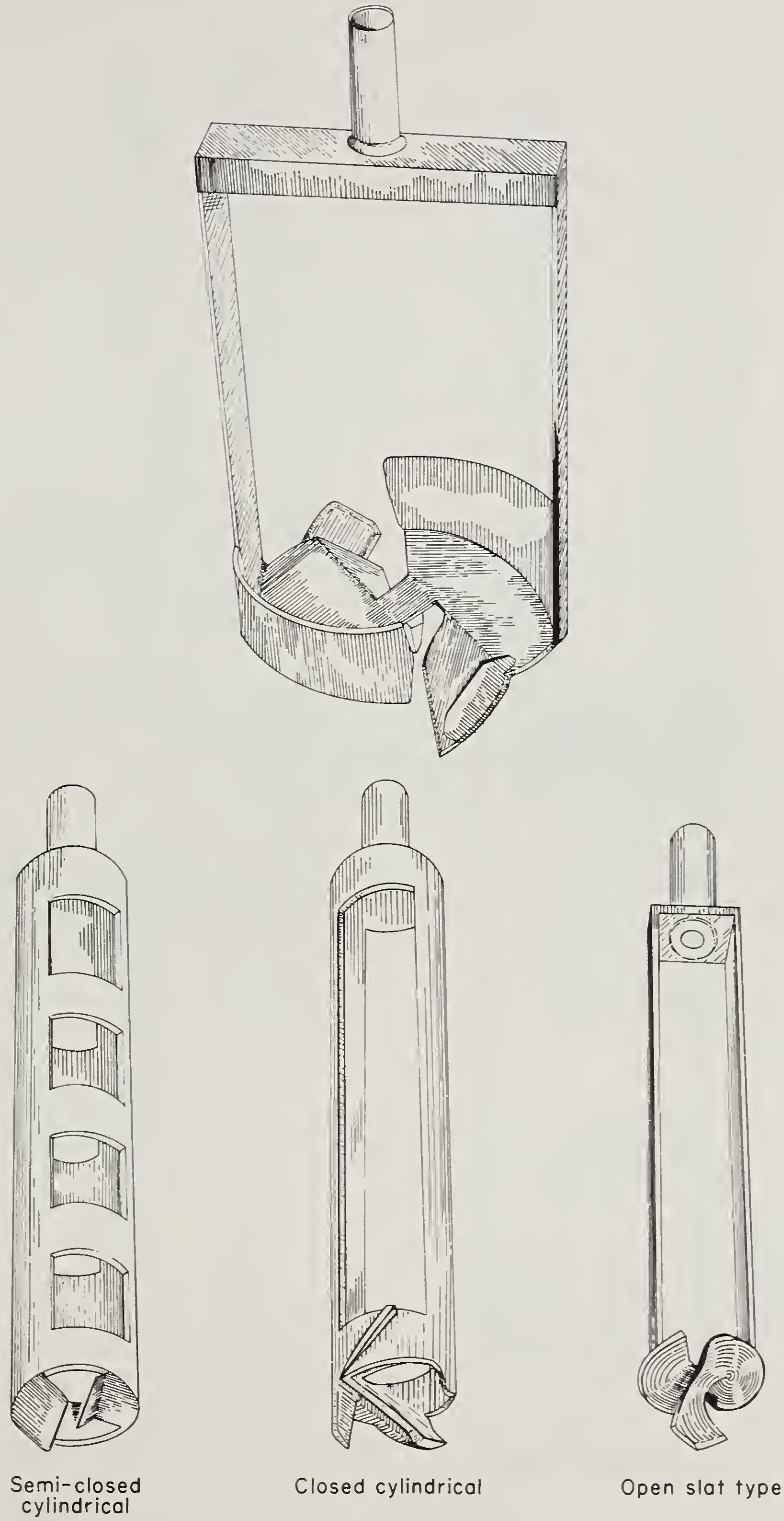


Figure 2-20 Various Types of Bucket-Auger Bits

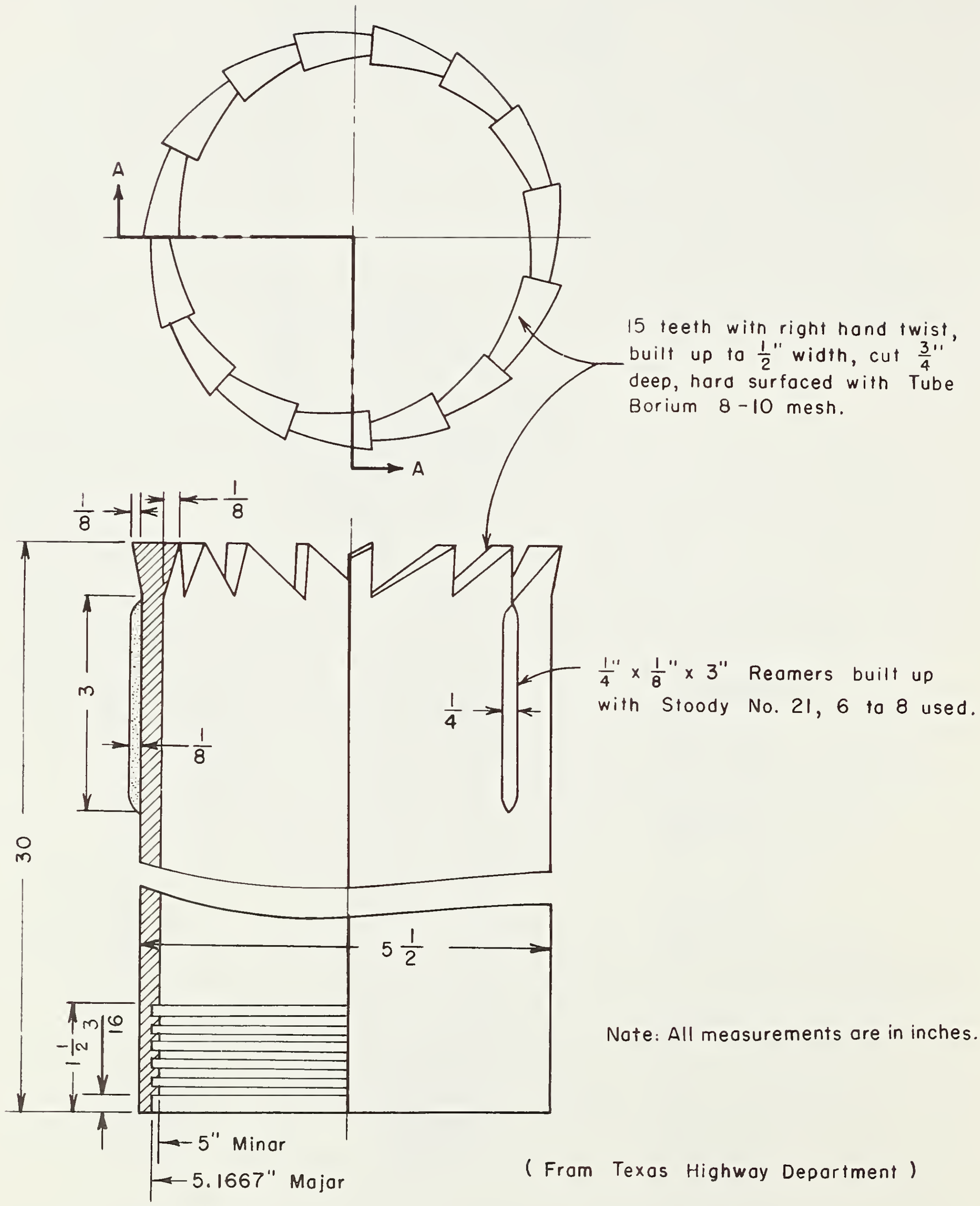


Figure 2-21 Dry Barrel Samples

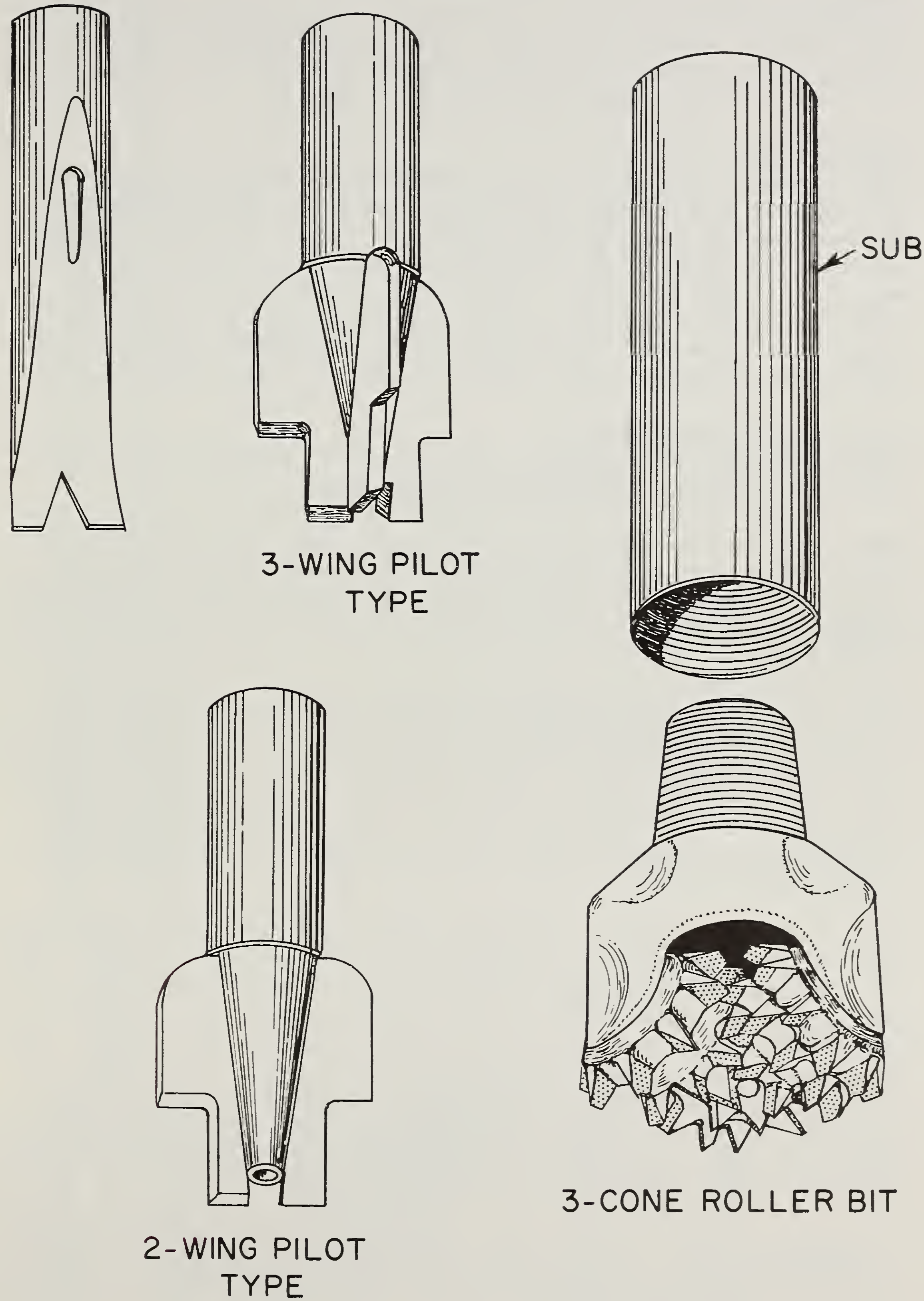


Figure 2-22 Various Types of Fishtail and Roller Bits

Other Drilling Equipment

General

In addition to the cutting and sampling tools outlined on the foregoing pages, certain additional equipment is needed to carry out boring and sampling operations. Needed and optional miscellaneous equipment is described on the following pages.

Drive Hammers

Drive hammers (Figure 2-23) are required for driving casing and for conducting standard penetration tests. A 140-pound hammer is required for the standard penetration test and heavier hammers are used for driving and removing casing. They may be operated manually or automatically. All drill rigs should be equipped, at least, with the 140-pound hammer. Where it is anticipated that considerable driving of casing might be necessary, an additional heavier drive hammer will result in more efficient operations.

Drill Rod and Couplings

Table 2-10 lists the various sizes of drill rods and couplings which are normally recommended for SCS work.

Many types of tool joints and drill rod couplings are available. Some types, such as those with tapered thread joints, are more subject to wear and to splitting of the female joint or box than others. The non-tapered flush joint, three threads per inch, is satisfactory.

Drill rod is normally manufactured with a box on each end. It is good practice to use a drill rod adapter or sub on each end. The female connection is made by means of a pin to box sub screwed into one end of the pipe. The male connection is made by means of a pin to pin sub screwed into the other end. When the subs become worn or damaged, they can be replaced at less cost than replacing the entire section of drill rod.

Drill rod not exceeding ten-foot length (ten feet, six inches with subs on both ends) is best adapted for site investigations. Twenty-foot lengths of drill rod are cumbersome to handle and transport. It is desirable for the mast of a rotary drill to be of sufficient height to handle at least two ten-foot lengths with subs (21 feet). This greatly expedites placing and removing tools from deep bore holes. The number of sections needed for a particular drilling rig will depend on the maximum number of feet which might be bored with the rig (seldom more than 100 feet), plus several spare rods for replacement in event of damage.

Solid drill rods are used with power augers where water circulation is not used. Flush joint drill rod of standard "AW" size, 1-3/4 inch outside diameter, or "BW" size is satisfactory for most power auger operations.

Since the joints need not be watertight, a simple joint such as a square pin and box, connected by a drive pin, is satisfactory.

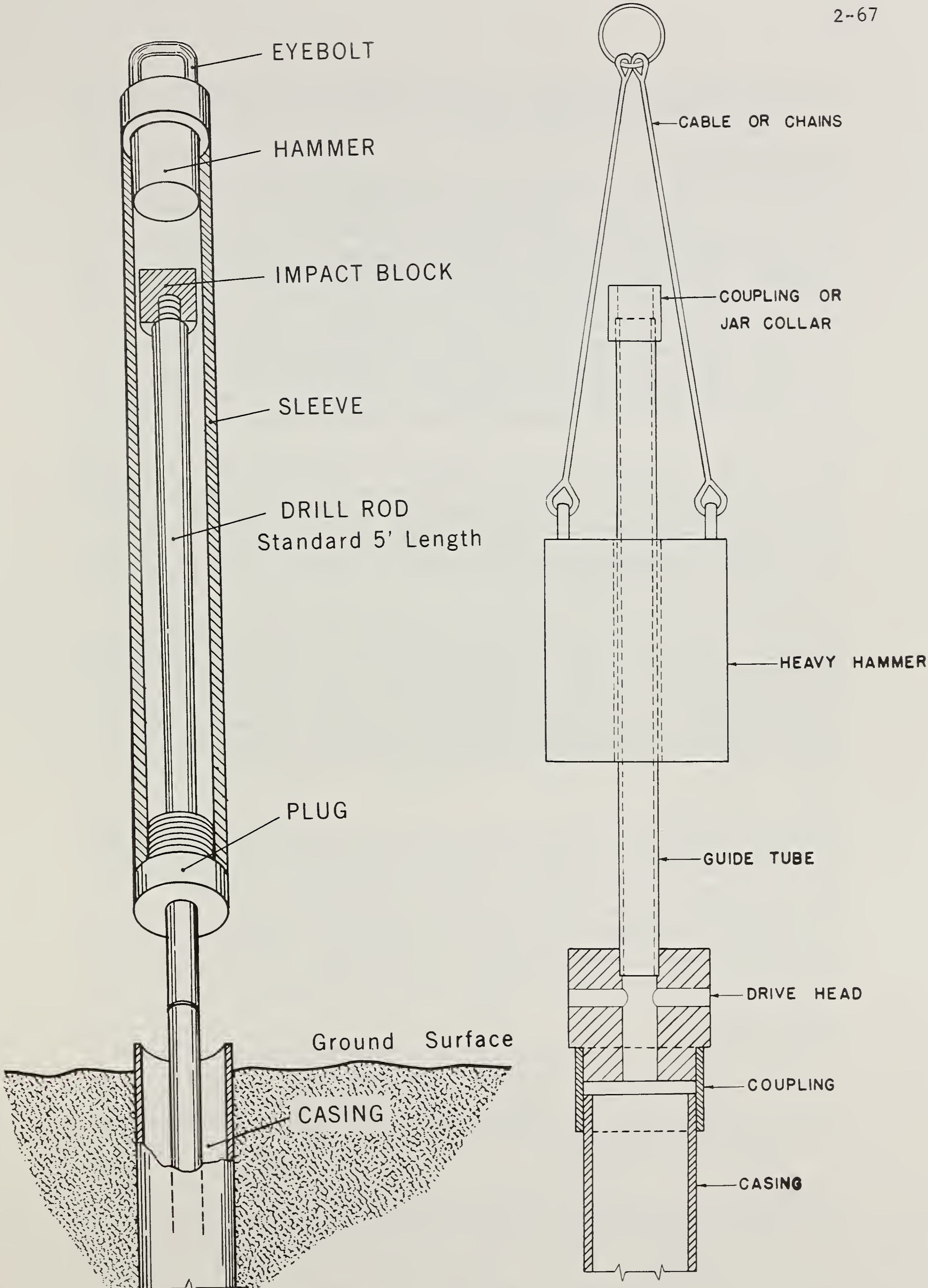


Figure 2-23 Safety and Casing Drive Hammers

Normally three 10-foot, two 5-foot, and two 2-foot lengths of drill rod will be ample for augering.

Clean-Out Tools
Clean-out tools are necessary for cleaning out bore holes preparatory to sampling. Materials which contaminate the sample or which may damage the sample must be removed. The type of tool and method required depends upon the nature of material to be removed. Certain types of cutting tools used for advancing holes such as barrel and bucket augers, leave relatively clean dry holes. Other types may leave gravel, slurry, water, drilling mud and other material in the hole. When the drilling hole contains slurry and drilling mud only, these materials may be removed by washing and flushing. Other types of materials require special tools. Some of these are described as follows:

Hole bailer - A hole bailer is useful for removing water and fine slurry. Its principal use is to remove water in the measurements of groundwater level. The outside diameter of the bailer should be one inch smaller than the hole diameter and preferably five feet long. It should have a dart bottom valve. A bailer of this type is shown in Figure 2-24.

Clean-out tools - Hole-cleaning tools perform the function of removing loose soil and rock particles from the bottom of a hole before samples are taken. Various types of cleaning tools have been designed to operate in various soil conditions. These tools include jetting wash pipe with scrapers, clean out jet augers, and sludge barrels. Some of these are illustrated in Figure 2-24.

Sample Catch Pan
A sample catch pan is needed when augering with either a rotary drilling rig or power auger. When the auger bit is raised, the pan is placed under it and the soil knocked into it. The material from the pan is then dumped in piles representing 1.0 to 2.0 foot increments for visual inspection. Normally this pan should measure about two feet by two feet. It may be fabricated locally.

Barrel Rack
A barrel rack will also be helpful to remove cores from double-tube soil and rock core barrels. A typical rack of this type is shown in Figure 2-25. It should be designed to fit the sizes of barrels being used. It is not available on the market and must be specially fabricated.

Slush Pits
Portable, or tank-type, slush pits are sold by some drilling equipment companies for use with rotary drill rigs. They may be needed where hard rock or gravel beds prohibit digging of adequate slush pits. Each time the drilling rig is moved, the mud and water must be removed from the tank to make it light

enough to move. When possible, it is easier and faster to dig a small pit or sump for the drilling fluid at each hole. This is particularly true for shallow drilling and frequent moving from hole to hole.

Miscellaneous Equipment

There are many additional tools and pieces of equipment which facilitate drilling operations. Some are optional while others are necessary to carry out certain cutting and sampling procedures. The uses of some tools such as boulder busters, fishing and recovery tools, water swivels, foot clamps, and pulling plates are quite obvious and need no further description. (See Figure 2-26.) Some additional equipment which may be useful in expediting site investigations are described briefly in the following section.

Water And Tool Trucks

If the source of water is remote from the site, considerable drilling time may be lost by continual interruptions for replacement of water. Where it is anticipated that such conditions may be encountered repeatedly, it may be advantageous to have a separate water and tool truck.

Mobile Trailer

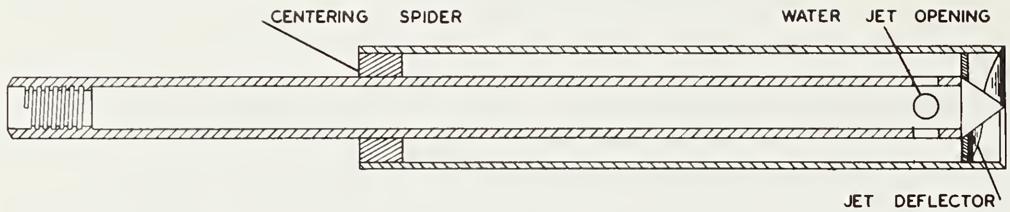
Drilling parties may be equipped with a mobile trailer or field office. These trailers should be equipped with desk, filing cabinet, butane heating stove, butane hot plate for melting wax and drying samples, and cabinets in which diamond bits and other equipment may be kept.

Miscellaneous Items

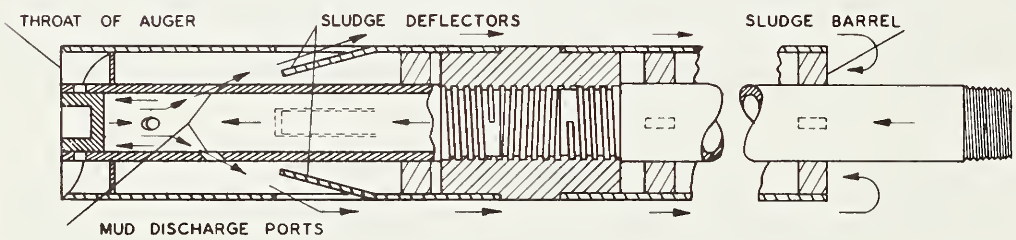
Other items which may be useful include testing kits for gypsum, clacium carbonate, montmorillonite, or other minerals, a hand level, sieves, a Brunton compass, and geologic maps and literature on the area. A dumpy level, level rod, and aneroid barometer may also be useful. A mirror is useful in reflecting sunlight into drill holes. Oxyacetylene welding equipment is very useful for periodic rebuilding of bits with the borium or other hard alloys, and for general field repairs to the drilling equipment and vehicles.

Drill Rigs

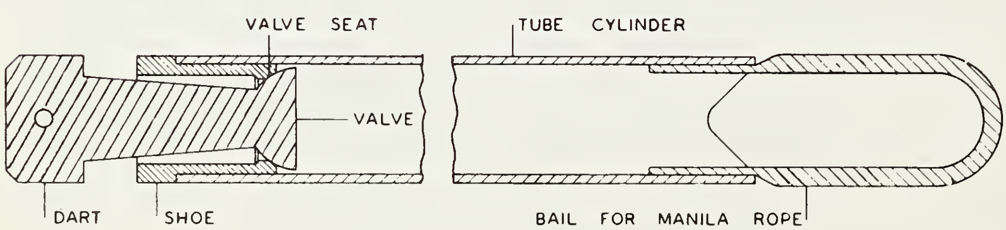
The term "drill rig" implies all of the equipment necessary to operate cutting and sampling tools. In general, it embodies such items as the power unit, derrick, drillhead, draw works, transmission, controls, and other appurtenances. Drill rigs may be truck, trailer, or skid-mounted. (See Figure 2-27.



STANDARD CLEANOUT AUGER



CLEANOUT AUGER
WITH SLUDGE BARREL



DART VALVE BAILER

Figure 2-24 Hole Bailer and Clean-Out Augers

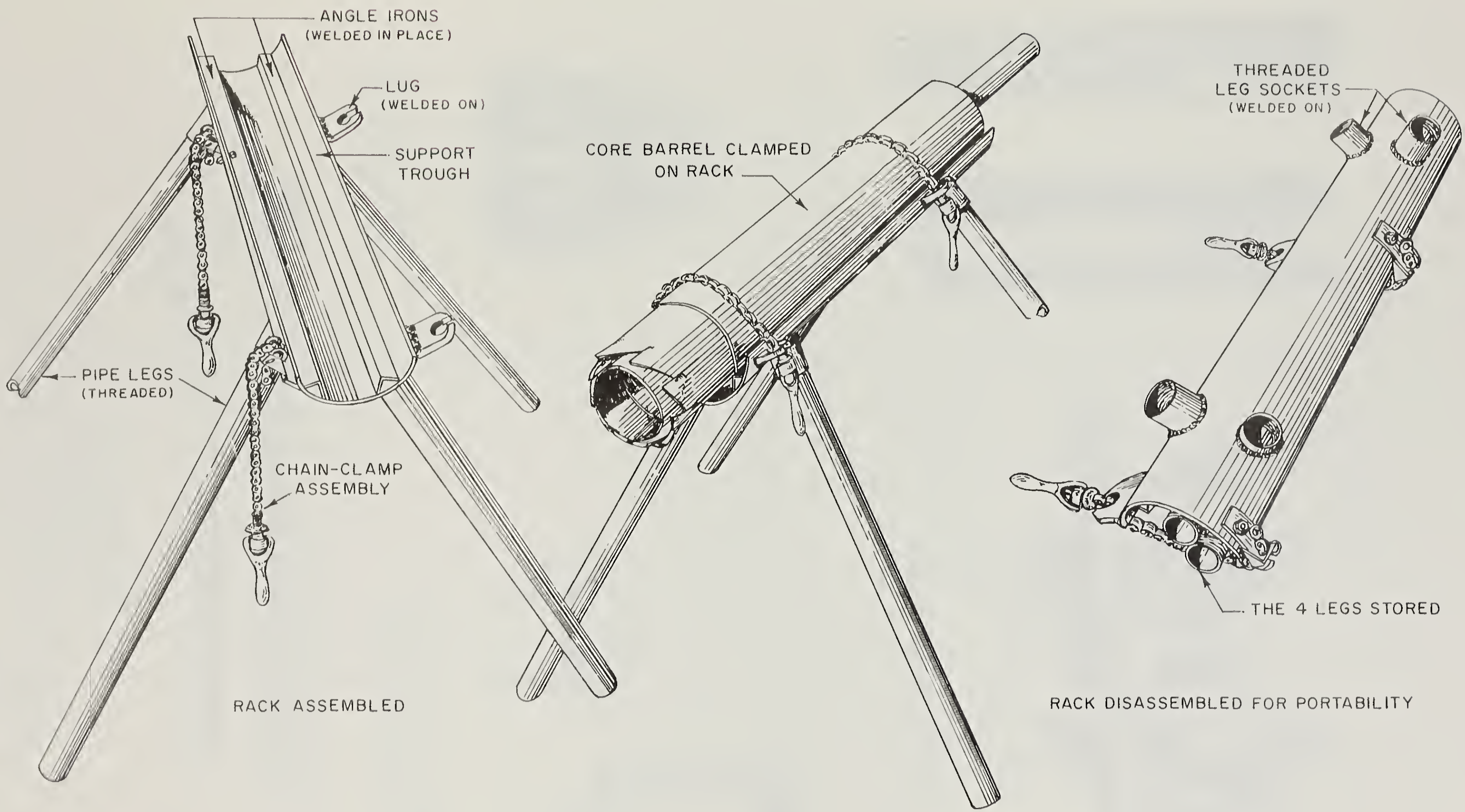
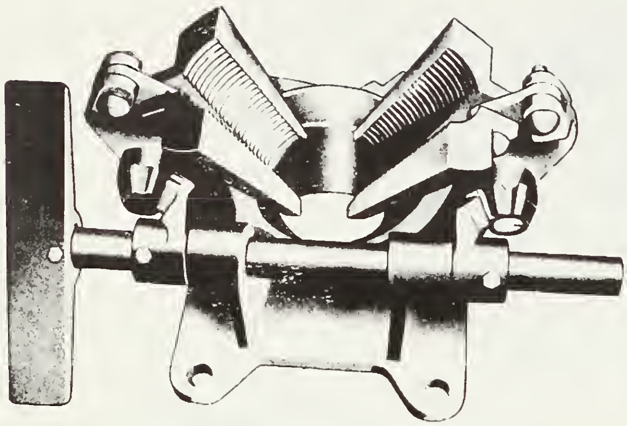
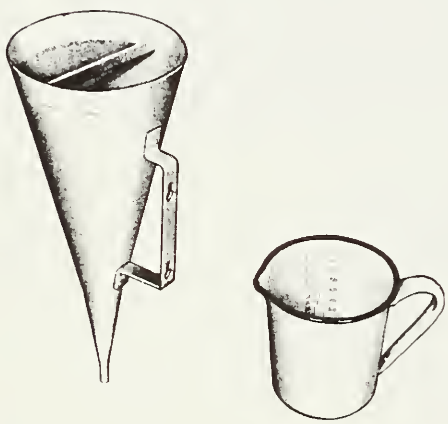


Figure 2-25 Barrel Rack with Barrel in Place



Safety foot clamp

Drill rod taps

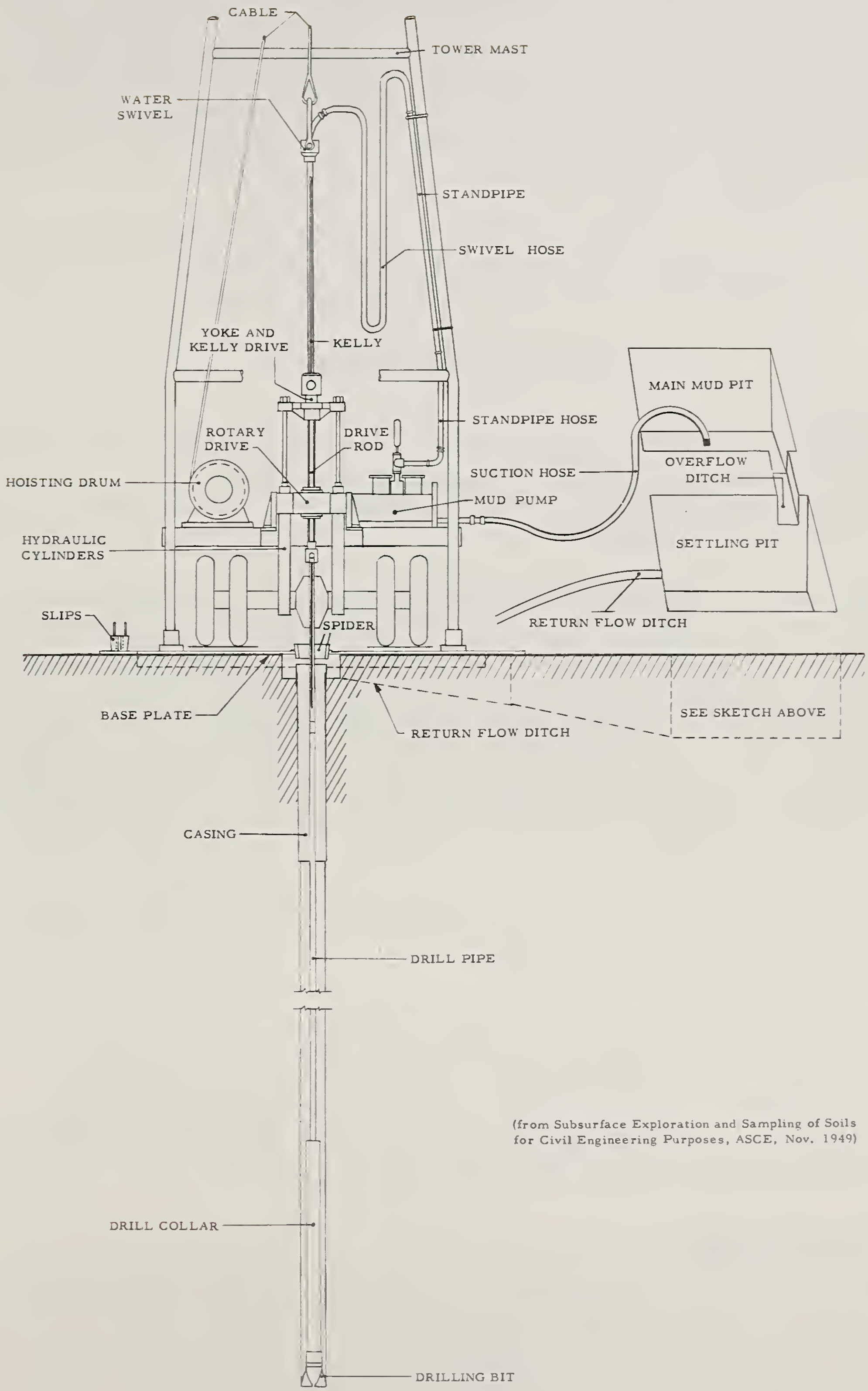


Marsh funnel and
graduated cup

Core catcher
(spring type)

Holding irons

Figure 2-26 Miscellaneous Equipment



(from Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, ASCE, Nov. 1949)

Figure 2-27 Rotary Drilling Rig

There are many types of drill rigs available on the open market. Each is capable of performing certain kinds of work. Most can be augmented in respect to component parts and mounting. The selection of the proper drill rig to carry out a particular work load depends on numerous conditions. These include the anticipated work load in respect to size and purpose of proposed structures; the nature of site conditions in respect to character of materials to be bored, depth of boring, and sample requirements; terrain; and other conditions.

Guidance in the selection of adequate drill rigs to meet conditions to be encountered in the work load of a particular state may be obtained from the E&WP unit serving that state.

Stabilizing Bore Holes

Casing

Temporary casing or lining of the bore hole with steel pipe is the most positive method of stabilizing a bore hole. It is normally required for certain methods of advancing holes such as percussion drilling.

Many types of standard and special pipe are used as casing. Recessed outside couplings provide the strongest joint and are commonly used in soils exploration. An open joint is used under normal conditions, but a butt joint is often preferred when the casing is to be driven through hard ground or ahead of the boring. Repeated use will damage the threads of open joints and cause beading and upsetting of butt joints. Flush jointed casing has a smaller resistance to driving and withdrawal than casing with outside coupling.

The lower end of the casing is generally protected by a casing shoe of hardened steel, with an inside bevel so that displaced materials will be forced into the pipe.

Because of the expense and time consumed in casing rotary core drill holes, it normally is not used where holes can be stabilized with drilling mud. Casing is required for the installation of relief wells. In this respect the purpose is to hold back wet, loose formations while the filter pack material is being placed around the well screen.

Drilling Fluids

An uncased shallow bore hole of the type usually drilled by rotary core drilling methods can ordinarily be stabilized with a properly proportioned drilling fluid or "mud" (Table 2-11). Drilling muds consist of highly colloidal gel-forming clays. Native clays may also be added to the drilling fluid. The drilling fluid forms a relatively impervious lining or "mud-cake" on the side walls of the bore hole. Weighting materials such as ground barite may be added to the drilling fluid to

increase its specific gravity and prevent caving of the hole in troublesome soils or when the fluid must carry very coarse-grained materials in suspension. Two slush pits should usually be dug downslope from the drill hole, with a small channel connecting them and the drill hole. The drilling fluid will then flow from the hole to the first pit, where the coarse material will settle out, and then to the second pit, where the mud can be picked up for recirculation (see Figure 2-27). Sand should be removed from the pits as it accumulates. If this is not done, the pump and the sampler may become clogged with sand. If coarse material is not present, one slush pit may suffice.

When using drilling fluids, the pump pressure and discharge should be hand controlled for soils and soft erodible rock so that the rate of circulation of drilling fluid can be controlled independently of bit speed. If the rate of circulation is too slow, or the bit pressure too great, the bit and fluid passages will plug. If the flow is too great (this is often the case), erosion of the core and soil below the bit will result. In the latter case, the drilling fluid may be forced to seek a path inside the cutting shoe, alongside the soil core, and through the vents, thereby eroding or removing part of the core. Generally, the pump pressure should be the minimum amount necessary to circulate the mud freely and carry the cuttings from the hole.

Pumps should have a control for regulating water pressure. If the pump has a separate power plant, pump pressure can be adjusted independently of bit speed.

Rigs which have a common power plant to turn the bit and run the pump, should be equipped to bypass part of the fluid, and reduce flow to the sampler.

Table 2-11 Approximate Proportions of Mud Mixtures 1/

Purpose of drilling mud	Approximate proportions of material per barrel of water 2/	Viscosity 3/	Descriptive consistency
Assisting cutting operations by the sampler	10 to 30 pounds of bentonite	Variable as needed	Variable as needed
For lifting cuttings from hole	10 to 15 pounds of bentonite for fine-grained soils	Slightly higher than water	Thin cream
	30 pounds of bentonite for coarse-grained material	About 1.3 times the viscosity of water	Very thick cream
For supporting the drill hole	30 pounds of bentonite and 5 pounds of barite	About 1.3 times the viscosity of water	Very thick cream
For assisting to hold the sample in the sampler	10 to 30 pounds of bentonite and 0 to 10 pounds of barite	Slightly higher than water to 1.3 times the viscosity of water	Thin cream to very thick cream

1/ (U.S.B.R. Earth Manual, Tentative Edition with Revisions, 1951, 1958).

2/ One barrel equal to 50 gallons.

3/ Viscosity is measured by a Marsh funnel (see Figure 2-26) which is calibrated with water at 72° F. The time required for a given amount of water to flow through the funnel is considered as 1.0. The value listed above is the relative time for the same amount of mud mixture to flow through the funnel.

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1/ underlining indicates definition or most important reference

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